Evaluation of the Effect of Recycled Asphalt Shingles on Ontario Hot Mix Pavement

by

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AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

Abstract

Due to the 15-20 year life span of roofing shingles, 1.5 million tonnes of asphalt roofing shingles are being demolished and replaced annually in Canada from both residential and commercial facilities. These roofing shingles are manufactured from very high quality materials which are considered a valuable by-product. Recycled Asphalt Shingles (RAS), a product containing approximately 30% asphalt cement by mass, is a valuable additive to Hot Mix Asphalt (HMA) pavements and a potential savings for the construction industry.

Recycling of demolished asphalt shingles is a significant new step forward in abating the need to put the waste into landfills. This re-use creates a great opportunity in reducing materials being dumped at landfills while providing an additive to HMA mixtures for paving. Therefore, this leads to economic, environmental, and social benefits for all the stakeholders and road users such as reduced need for landfill space, conservation of virgin materials and environment, and financial saving.

The research involved evaluating the use of demolished shingles in six typical Ontario Hot Mix Asphalt (HMA) mixtures; HL 3 (1.5% RAS, 13.5% RAP), binder layer mixes SP19 (6% RAS, and 3% RAS, 25% RAP), surface layer mixes SP12.5 FC 1(3% RAS, 17% RAP) and SP12.5 FC2 (6% RAS and 3% RAS, 12% RAP). The six HMA mixes were also designed to contain Recycled Asphalt Pavement (RAP). This further complicated the research as both RAP and RAS were added. All mixes were designed and tested at CPATT laboratory; in addition a test section was paved at the CPATT Test Track.

This research involved both laboratory and field evaluations of mixes containing RAS to develop pavement performance modeling for all six mixes using the updated Mechanistic-Empirical Pavement Design Guide (MEPDG). A life-cycle assessment of the six HMA mixes was performed to quantify the environmental impacts using the Pavement Life-Cycle Assessment Tool for Environmental and Economic Effects (PaLATE) and rigorous economic costs/benefits were assessed using Life Cycle Cost Analysis (LCCA). Calibrations of models for Ontario conditions were completed. Test slabs were also constructed to simulate climatic changes by running freeze-thaw cycles based on weather data over the past ten years.

Three field test sections located in the Town of Markham and one at the CPATT Test Track were monitored and assessed under as part of the research. Regular pavement condition assessments were carried out on all the test sections by performing non-destructive tests using a Portable Falling Weight Deflectometer (PFWD) and distress survey in accordance with the Ministry of Transportation (MTO) guidelines. The CPATT Test Track was evaluated with both the PFWD and surface distresses, whereas

only distress surveys were performed on the three residential streets in the Town of Markham. The evaluations demonstrated that the pavements were in good conditions throughout the monitoring period of the research (four years for the three residential streets in the Town of Markham and two years for the CPATT Test Track).

The structural analysis using the MEPDG indicated that Mix 3: SP19 3% RAS and 25% RAP had the best performance followed by Mix 2: SP19 6% RAS when considering all factors in the Life-Cycle Assessment. Mix 3 exhibited maximum savings on environmental emissions, energy and water usage, best adoptability to climatic change and skid resistance properties with minimal life cycle costs.

The pavement performance and life-cycle assessment modeling demonstrated encouraging results for the use of RAS in HMA pavements from which guidelines were developed for its use. It is important to note that careful mix design should be carried out when RAS is added to HMA especially when RAP is also used. This includes measuring of all key properties especially at low and high temperatures. In short, RAS can be a valuable additive in both surface and binder layers of HMA pavements. It provides an environmentally friendly and cost-effective innovation for the Ontario paving industry and can be considered for usage elsewhere with appropriate engineering.

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Last but not least, special thanks to my Lord, Savior, love, and best friend Jesus Christ. For always encouraging me to never give up and reminding me that all things are possible as long as we believe and patiently set our minds to accomplishing the desired goal.

Dedication

I would like to dedicate this thesis to my parents (who sacrificed the joy of parenthood for us to attain an education and hope of a better future), family, Keith and Elizabeth Harris (who have been like parents to me throughout my life), the Rotary Clubs of Dundas and Caledonia, and friends for supporting me throughout my academic years and professional career.

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Chapter 1: Introduction

1.1 Background

Millions of tonnes of asphalt roofing shingles are generated annually for residential and commercial purposes. However; at the end of their 15 – 20 year design life, the majority of these shingles end up in landfills [Tighe 2008]. Currently, as demonstrated by the natural resources of Canada study in 2007 “Enhancing the Recovery of End of-Life Roofing Materials”, 1.5 million tonnes of demolished roofing shingles are annually generated in Canada. These shingles are manufactured from high quality material and although they may need to be replaced as roofing material, these still can be considered a valuable by-product. If the shingles can be processed and tested to ensure quality, they can be engineered into asphalt pavements.

For more than two decades, manufactured shingles in asphalt pavements have been used. These shingles are new and have been determined to be defective for use on a roof. However, they can be shredded and graded for use in asphalt pavements [McGraw 2010]. The use of demolished asphalt roofing shingles that have been in service on buildings or roves is a recent research area. Recycled Asphalt Shingles (RAS) are grouped into two types; organic shingles and fiberglass shingles. The typical composition of the shingles is as given in Table1-1.

Table 1-1: Typical Composition of the Shingles [CIWMB 2007]

Component	Organic Shingles	Fiberglass Shingles
Asphalt Cement	30% - 36%	19% - 22%
Felt	2% - 15%	2% - 15%
Mineral granules/Aggregate	20% - 38%	20% - 38%
Mineral Filler/Stabilizer	8% - 40%	8% - 40%

In essence, this is a very high quality manufactured material and if it can be properly shredded, sorted and cleaned to remove contaminants, graded and tested; it can provide many benefits as it stiffens the mix making it more resistant to traffic loading. Furthermore, given the need to consider sustainability, shingles can be recycled instead of ending up in a landfill. Recent research indicate that RAS can improve pavement performance by increasing resistance to wear and moisture, and decreasing deformation, rutting, thermal fatigue and cracking [Shingles 2007].

1.2 Introduction

The Centre of Pavement and Transportation Technology (CPATT) at the University of Waterloo has partnered with Miller Paving Limited and the Ontario Centre of Excellence (OCE) to carry out a study evaluating the performance of RAS in typical Ontario asphalt mixes. As part of the research project, a 420 metre long test section was paved using RAS in Hot Mix Asphalt (HMA) HL 3 at the CPATT test track located in the Regional Municipality of Waterloo Waste Management Facility. This newly added additional test section complements the existing test sections which consist of five asphalt pavement test sections, four concrete pavement test sections and four interlocking concrete pavers test sections. The mix design of the CPATT Test Track section containing RAS was designed by Miller Paving Limited. Steed and Evans Limited placed the test section on October 19, 2009 and October 20, 2009 under the Region of Waterloo paving contract.

The pavement performance of RAS in HMA pavement was evaluated through an integrated laboratory study and field study. Life cycle cost and environmental impacts analysis was also performed. The two test locations included in the field evaluation were;

- The CPATT Test Track, which is a 420 metre HL 3 test section containing 1.5% RAS and 13.5% Reclaimed Asphalt Pavement (RAP) constructed in October 2009 and located in the south-east corner of the Regional Municipality of Waterloo's Waste Management Facility. The Test track was constructed as an access road to the various landfill cells.
- The Town of Markham test location consists of three test sections under study namely: Ida street (SP12.5 FC1 3.5% RAS), Paul Street and Vintage Lane (SP12.5 1.5% RAS and 13.5% RAP), and Thornhill Summit Drive (SP12.5 1.5% RAS and 13.5% RAP). These are low volume residential streets with the pavement placed by Miller Paving Limited in 2007 and provide some insight into field performance of pavement with HMA containing RAS and/or RAP after a few years in service.

The laboratory research involved simulating field pavements of various percentages of RAS and/or RAP in six typical Ontario HMA namely, Mix 1: HL 3 1.5% RAS and 13.5% RAP, Mix 2: SP19 3%

RAS, Mix 3: SP19 3% RAS and 25% RAP, Mix 4: SP12.5 FC1 3% RAS and 17% RAP, Mix 5: SP12.5 FC2 6% RAS and Mix 6: SP12.5 FC2 3% RAS and 12% RAP.

Several laboratory Slabs were constructed to evaluate their freeze thaw performance. The research involved the new Canadian pavement design model, the Mechanistic-Empirical Pavement Design guides (MEPDG) to evaluate RAS in HMA pavements. Also it involved the development of appropriate Life Cycle Cost Analysis (LCCA) Ontario RAS parameters, evaluation of RAS using the Pavement Life-Cycle Assessment Tool for Environmental and Economic Effects (PaLATE), and Ontario field pavement condition assessment.

The study involved development and evaluation of the benefits and costs of HMA that uses RAS in terms of economic, environmental and technical aspects in the Ontario context. However, it is notable that the evaluation could also be applied to other Canadian jurisdictions. Guidelines for the usage of RAS in Ontario HMA were developed. Using the newly developed MEPDG, the performance of RAS in HMA was evaluated in terms of roughness, rut-depth, thermal cracking, fatigue cracking and permanent deformation or rutting. The sustainability of using RAS in terms of environmental metrics and cost has been evaluated in the Ontario context.

1.3 Purpose of the Research

The purpose of the research is to evaluate the performance and potential costs/benefits of RAS in Ontario Hot Mix Asphalt (HMA) pavements through an integrated laboratory and field study. The specific objectives of the research were to;

1. Perform structural analysis of RAS HMA pavements using MEPDG.
2. Carry out a cost and sustainability analysis using LCCA and PaLATE analyses of using RAS in Ontario HMA pavements.
3. Evaluate pavement performance through laboratory freeze-thaw test slab and field test section performance using visual distress survey and standard test methods.
4. Evaluate and validate the sustainability metrics of using RAS in HMA pavements through a performance model, LCCA model, and PaLATE model.
5. Develop guidelines for the usage of RAS in HMA pavements.

1.4 Research Methodology

In order to achieve the research objectives, the following tasks were chronologically carried out:

Task 1: Literature Review

Review of previous works carried out on using RAS within North America as well as related pavement performance analysis. The literature review identified key findings of the previous work and identifies the gaps in existing practice. It also provided a context for usage of RAS in Ontario pavements.

Task 2: Laboratory Evaluation of RAS HMA

Eighteen HMA test slabs (three slabs per mix) were constructed with various percentages of RAS for the purpose of freeze thaw cycling, performance analysis, and validation of the results. In total six different HMA containing various percentages of RAS and/or RAP were evaluated as well as Control Mix namely;

- Control Mix: Conventional HL 3
- Mix 1 – HL 3 1.5% RAS and 13.5% RAP,
- Mix 2 – SP 19 6% RAS
- Mix 3 – SP 19 3% RAS and 25% RAP
- Mix 4 – SP 12.5 FC1 3% RAS and 17% RAP
- Mix 5 – SP 12.5 FC2 6% RAS
- Mix 6 – SP 12.5 FC2 3% RAS and 12% RAP

The final mix design for each of these mixes was determined through an optimization process.

The initial slab surface characteristics were determined using the sand patch method to determine the surface texture and the British Pendulum to assess the friction properties of the slab. A visual analysis was also conducted to benchmark performance before and after carrying out freeze-thaw cycling.

Task 3: Field Test Sections Evaluation and Assessment

A total of four test sites, three in the Town of Markham and one at the Centre for Pavement and Transportation Technology Test Track were constructed and evaluated. The laboratory data were compared to the observed field performance at the four sites.

Task 4: Calibration of Pavement Performance Modeling

The MEPDG is currently being adopted for usage in Canada by all provincial transportation agencies and should be implemented by 2013. This research involved evaluating the usage of the MEPDG and its applicability with RAS HMA for Ontario pavements. The structural performance, maintenance, rehabilitation, and reconstruction were modeled using laboratory and field data. Level 1 or the most specific protocol of the design procedure was used to characterize each asphalt mix. This involved assessing material properties from previous CPATT research and using the data from Task 2 and Task 3 of the current research.

The pavement performance results from MEPDG were validated in the Ontario context by comparing with conventional HMA using statistical analysis.

Task 5: Calibration of Life-Cycle Assessment (LCA)

LCCA involved comparing several alternative Ontario HMA pavement designs and associated maintenance strategies used by cost estimations, and pavement treatment strategies. The Pavement Life-Cycle Assessment tool (PaLATE) determines the amount of emission associated with the various design and provides sustainability metrics. The combination of both assessments (LCCA and PaLATE) provides a decision making tool.

The results from LCA were validated in the Ontario context by comparing the mixes with conventional HMA using statistical analysis.

Task 6: Development of Guidelines of using RAS in HMA pavements

Guidelines were developed considering all the factors for the usage of RAP and/or RAS in HMA pavements. Recommendations were drawn for further study.

The research methodology of the study is summarized in Figure 1-1.

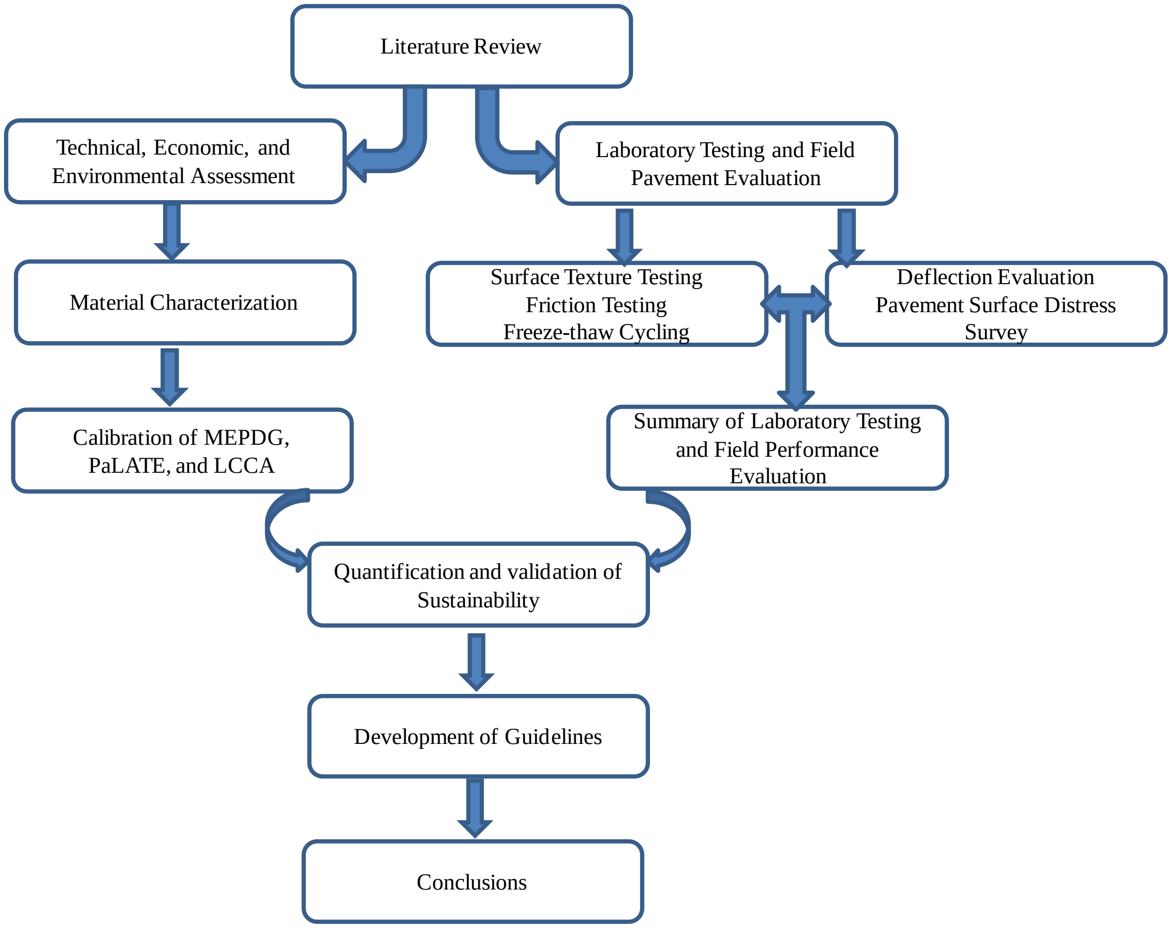


Figure 1-1: Research Methodology for Evaluation of RAS in HMA

1.5 Organization of Thesis

This thesis is divided into seven chapters as follows;

Chapter One: Introduction; provides a general overview of the research, objectives, scope and research methodology.

Chapter Two: Literature Review; this chapter details aspects of RAP and/or RAS in HMA pavements including definitions, laboratory testing and field evaluations, performance analysis models for the mixes and life-cycle assessments to quantify the sustainability and economic benefits of the mixes for Ontario HMA pavements. This was completed through analyzing previous research on RAS including history and evolution of usage, testing and performance in HMA pavements. This was used to identify existing gaps and research needs.

Chapter Three: Research Methodology; this chapter discusses the research approach to achieve the research objectives. It includes details of the laboratory testing, field evaluations, pavement performance prediction modeling and life-cycle assessment (PaLATE) and LCCA modeling.

Chapter Four: Laboratory and Field Pavement Evaluation; this chapter discusses the field and laboratory pavement assessment of the RAP and/or RAS usage in HMA pavements. The four test sections were evaluated in accordance with MTO guidelines for any surface distresses and/or pavement deformations. This provided insights into the performance of RAP and/or RAS in HMA pavements.

Chapter Five: Structural Evaluation of RAS Pavement; this chapter discusses the material characterization, and performance prediction analysis using the M-EPDG. Both the laboratory and field data were used in the calibration of the design guide for Ontario HMA pavements however; suggestions on how this can be adapted to other Canadian provinces and/or areas were identified.

Chapter Six: Life-Cycle Assessment of RAS Pavements; this chapter quantified the sustainability of using RAP and/or RAS in HMA pavements and the economics associated with the mixes. The environmental impact assessment was carried out through the use of an excel-based spreadsheet PaLATE and the economic assessment through Life-Cycle Cost Analysis (LCCA).

Chapter Seven: Conclusions and Recommendations; this chapter provides the conclusions for the usage of RAP and/or RAS in HMA pavements. It also provides recommendations for further work.

Chapter 2: Literature Review

2.1 Introduction

All sectors of the transportation industry have been developing keen interest in the engineered incorporation of the use of Recycled Asphalt Shingles (RAS) in Hot-Mixed Asphalt (HMA) pavement mixtures. The use of newly manufactured RAS in HMA has been researched and slowly implemented in some areas of North America. Research has shown that incorporating 5% or less of manufactured shingle waste asphalt binder to total binder content in HMA does not significantly affect pavement performance [Austin 2011]. The performance of HMA with RAS has shown some benefits to agencies due to the increased asphalt and cement prices. Researchers have been able to demonstrate acceptable pavement performance using sustainable supplement to the HMA. Minnesota Department of Transportation (MnDOT) state is an example of a transportation agency around North America that has sponsored research for the past 15 years in an attempt to qualify and quantify the use of RAS in their HMA [McGraw 2010]. The state of Minnesota allows 5% as a maximum amount of shingle scrap in their HMA, by weight of aggregate.

2.2 Recycled Asphalt Shingles (RAS)

2.2.1 Introduction

Asphalt shingles as shown in Figure 2-1 compose two-thirds of the roofing market for both the residential homes and commercial roofing installations hence generating an estimated annual quantity of 7-10 million tonnes (US) and 1.5million tonnes (Canada). 90-95% of roofing asphalt shingle waste come from residential roof replacement (demolished roofing asphalt shingle) and the remainder being leftover installation/production scrap [FHWA 1998]. Roofing asphalt shingles are estimated to have a design life of approximately 15-20 years, after which they end up in landfills. The asphalt shingles represent a major and non-degradable waste stream in the landfill hence the negative environmental impacts. Asphalt shingles contain four basic materials including stiff asphalt cement, felt, fine aggregate, and mineral filler as shown in Figure 2-2. All these components are high quality and must meet various specifications [Austin 2011].

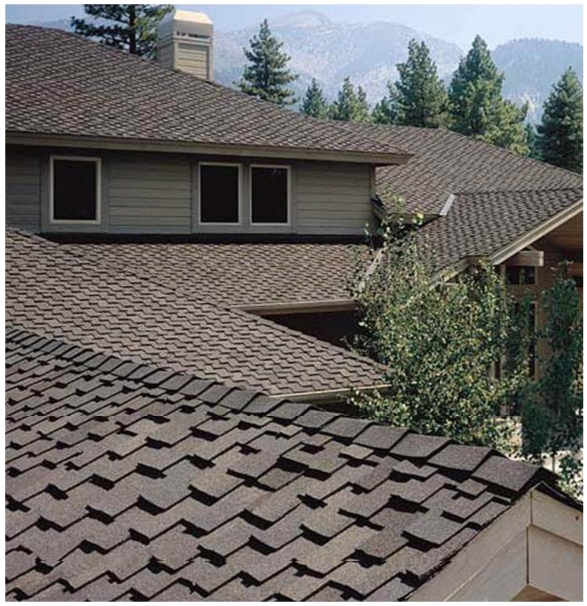


Figure 2-1: Residential Roofing with Asphalt Shingles [Mohammed 2008]

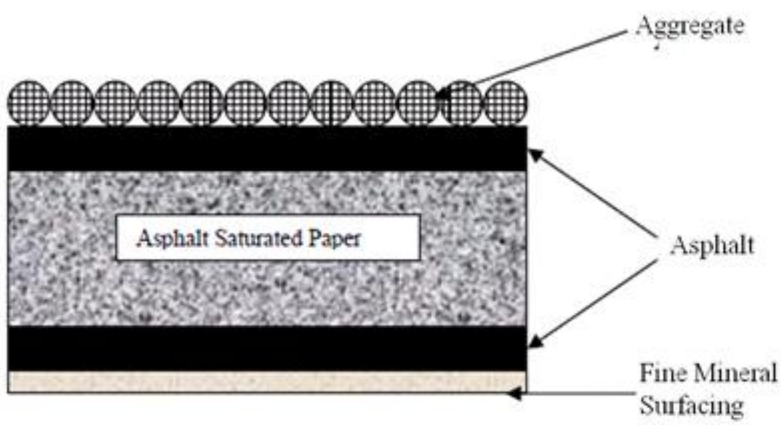


Figure 2-2: Profile View of a Typical Asphalt Shingles [Austin 2011]

For over two decades, the use of manufactured RAS in HMA has produced beneficial results for industrial pavement construction. Manufactured RAS are those shingles that are defective for usage on a roof and then recycled and incorporated into a pavement. However, the use of demolished roofing asphalt shingles is a new area of study. Depending on the amount of RAS incorporated in HMA, and the origin of RAS used, researchers have realized that there can be an improvement in pavement performance such as increased resistance to wear and moisture, and decreased deformation, rutting, and fatigue and cracking [Shingles 2007]. Figure 2-3 shows the demolished shingle and the finished processed product used in Hot Mix Asphalt (HMA) pavements.

Figure 2-3: Cleaned Demolished Roofing Asphalt Shingles and Finished Product [Seals 2010]

2.2.2 Types of Roofing Asphalt Shingles

Two types of shingles are manufactured in the roofing industry; (1) organic made from felt material saturated with tarpaper to improve strength and durability and (2) fiberglass made from glass particles cut into shingle shapes and coated with asphalt applied in a mat pattern to ensure a waterproof seal. Both types of shingle typically contain a higher percentage of asphalt cement per square metre, which can supplement virgin asphalt binder in HMA mixtures. The shingles consist of minus 4.75mm (No. 4 sieve size) particles as shown in Figure 2-3 supplementing fine aggregate fraction in HMA mixtures [FHWA 1998]. According to Foo 1999, a small percentage of RAS can displace a large amount of virgin aggregates.

Two kinds of shingles are added to the HMA pavement mixture, namely;

1. Manufactured roofing asphalt shingles, which has been widely incorporated in HMA mixes for over two decades. It is a newer material deemed unusable for roofing purposes and can be either straight from the manufacturer's plant or installation trimmings, which end up at landfill. The shingles contain no contaminants, and it is more uniform in content containing softer asphalt and more functional in HMA mixtures [Lum 2004].
2. Demolished/Tear-off roofing asphalt shingles (sometime referred to as post-consumer shingles) are the old shingles mostly from the residential homes after being in service for over 15 years during roof replacement which is contaminated with nails, wood and more deleterious materials.

The AC in this kind of shingles is significantly aged in that adding even a small amount in HMA can negatively affect pavement performance. Research needs to be done to bridge barrier to the use of post-consumer shingles that have been subjected to the natural conditions such as climatic effects. The older shingles contain higher percentages of asphalt cement than new shingles due to weathering and loss of aggregate overtime [Austin 2011].

The Asphalt Cement is hardened from oxidation and volatilization of lighter organic products, and contains fibres resulting in stiffer HMA mixes. Therefore, a softer asphalt cement binder should be used when blending with HMA. Demolished roofing asphalt shingles can adversely effect the moisture sensitivity of the HMA mix, hence strict adherence to specification and pavement design should be taken when designing RAS HMA pavements [FHWA 1998].

Due to the hardened asphalt binder in post-consumer shingles, it was observed that they exhibited easier workmanship characteristics such as easier to shred and less likely to stick together during processing [Austin 2011].

Recycled Asphalt Shingles (RAS) Engineering Properties

The stiffening influence of roofing shingle fibres improve the high temperature susceptibility and rut-resistance properties resulting in improved fatigue life in pavements with moderate increases in the RAS percentage. Using 3-5% RAS by mass of total mix results in denser compactive effort. This reduces the cold tensile strengths, and the resilient modulus results indicate no substantial effect in potential low-temperature cracking [FHWA 1998].

2.2.3 Potential Benefits from the Use of RAS in HMA

- RAS in new products reduces the negative environmental impacts associated with the extraction, transportation and processing of virgin materials,
- Conservation of landfill spaces (all non-degradable materials are diverted away)
- Reduces manufacturers and consumer costs due to reduced HMA production and contractor's disposal fees respectively.
- The asphalt shingle also binds the crushed stone granular together leading to effective dust control [shingle 2007].
- Reinforcement from fibers improves shear resistance to pavement cracking [Austin 2011].

- Shingle fibres and increased binder stiffness results in improvements in rutting and shoving resistance [CMRA 2010] [Austin 2011].

2.3 Reclaimed/Recycled Asphalt Pavement (RAP)

RAP is the term given to reprocessed pavement materials containing asphalt and aggregate obtained from reconstruction and resurfacing of asphalt pavements. RAP consists of high-quality well graded aggregates coated by asphalt cement [FHWA 1998].

RAP when properly crushed, screened, processed and graded into fine aggregates can be incorporated into hot mix asphalt and has demonstrated many advantages such as reduced construction costs, aggregate and asphalt binders conservation, existing pavement geometrics preservation, and energy conservation [Koch 2010].

In Ontario, the use of RAP on pavements is governed by highway specification OPSS 1150, which recommends its use from 20% to 40% by mass of total HMA mass. Both the high and low grade performance graded asphalt cement (PGAC) is recommended to be lowered by 6^0C.

2.4 Previous RAS Research

2.4.1 Highway 86, Waterloo, Ontario

The use of RAS in HMA pavements in Canada has been limited to the trial studies in Canada. In 1995, MTO constructed a section on Highway 86 in Waterloo, Ontario incorporating new manufactured shingles modifier in HMA. The asphalt pavement structures in these field trials are given in Table 2-1. Various field and laboratory evaluation were performed on the pavement structures by Lafarge North America Inc. with a final evaluation assessment in 2003.

Table 2-1: Summary of Asphalt Pavement Structure [Lum 2004]

SECTION	STATION		PAVEMENT STRUCTURE
	FROM	TO	
1	20+925	21+005	HL 1 / MDBC
2	21+005	21+575	HL 1 / MDBC (MSM)
3	21+575	21+995	HL 1 (MSM) / MDBC (MSM)
4	21+995	22+275	HL 1 (MSM) / MDBC

Note: HL = Hot Laid, MDBC = Medium Duty Binder Course, MSM = Manufactured Shingle Modifier

The study indicated no low temperature transverse and longitudinal cracks, raveling, fatigue cracking or rutting in the modified lane. It was observed to be in excellent condition after eight years of service [Lum 2004].

Another section of Highway 401 was resurfaced with 12.5mm SMA incorporating 3% MSM in 2001 and evaluated after three years' of service to assess its performance. The section was observed to be in good condition without any visual surface ravelling, low temperature cracks in transverse and longitudinal joints had reflected through the SMA pavement mixture [Lum 2004].

Lafarge also observed that it is easier to introduce an enhanced asphalt fibre binder modifier at the asphalt plant. The modifier improves rutting resistance and flexural strength. It exhibited an equal to if not better HMA durability [Lum 2004].

2.4.2 University of Waterloo's Centre for Pavement and Transportation Technology Study

CPATT partnered with Millers Group Inc, Ontario Centre for Excellence (OCE), Materials Manufacturing Ontario (MMO), and École de Technologie Superieure (ETS) in Montreal to study the effect of RAS on a Superpave 19C or Hot Laid 8 (HL 8) binder course in 2006 [Tighe 2008]. Five SP19C/HL 8 mixes containing various percentages of RAS and RAP were formulated in the CPATT laboratory in accordance with Canadian specifications for pavement design. Laboratory testing on the mixtures were carried out in dynamic modulus, resilient modulus (both by CPATT laboratory), and Thermal Stress Restrained Slab (TSRST) and French Wheel Rutting test (both by ETS laboratory). Dynamic modulus test predicts fatigue cracking and rut resistance, resilient modulus test provides an indication of fatigue and thermal cracking potential, TSRST assesses the thermal cracking resistance of the mix whereas the rutting test estimates the rutting susceptibility of the mix [Tighe 2008].

The various laboratory test results on HL 8 mixes are summarized in Table 2-2. Using a scale of 1 being the best while 5 was the worst in performance, the tests were ranked to ascertain the most optimal mix. The test results indicated that RAS and RAP could be incorporated into HL 8. Note this is a very basic HMA and based on these encouraging findings, it was desirable to examine other HMA mixes.

Table 2-2: Summary of the Laboratory Testing of the five HL 8 Mixes [Tighe 2008]

Mix Description	Dynamic Modulus	Resilient Modulus	Thermal Stress Restrained Slab Test (TSRST)	French Wheel Rutting Test
Mix 1 (Control): SP 19C, virgin materials	1	1	3	3
Mix 2: SP 19C 20% RAP	1	3	2	5
Mix 3: SP 19C 20% RAP, 1.4% RAS	2	2	1	4
Mix 4: SP 19C 20% RAP, 3.0% RAS	3	4	4	1
Mix 5: SP 19C 3.0% RAS	4	5	5	2

Overall, all mixes performed relatively well in the various laboratory tests. It was expected that there would be limited rutting in the field as all of the mixes displayed less than 4 mm of permanent deformation. The laboratory analysis indicated that Mix 3 was the optimum mix based on all test results when compared to Mix 4 or Mix 5 which also contained RAS [Tighe 2008].

2.4.3 Laboratory Testing of Vancouver HMA mixes containing RAS

Metro Vancouver conducted a study to evaluate alternative options of diverting solid waste from its landfills. The main objective of the research was to evaluate the feasibility of using HMA mixes incorporated with RAS without compromising pavement performance. Seven mixes were prepared including the control mix and were tested in the CPATT laboratory. These tests included dynamic modulus, resilient modulus, rutting resistance, fatigue endurance, and susceptibility to low temperature cracking. It should be noted that this study also used a rejuvenator for some of its mixes and evaluated the effect of the rejuvenator on the HMA mixes [Uzarowski 2010]. A summary description of the five mixes undertaken in the study for Metro Vancouver is given in Table 2-3.

Table 2-3: Summary Description of the Mixes [Uzarowski 2010]

Mix	Virgin Aggregate (%)	RAP (%)	RAS (%)	*Rejuvenator (%)
1	100	0	0	0
2	85	15	0	0.3
2B	85	15	0	0
3	97	0	3	0.3
4	95	0	5	0.3
5	82	15	3	0.3
6	80	15	5	0.3

[*] Rejuvenator was added at a rate of 0.3% of Asphalt Cement

The shingles were ground to 6mm – 7mm chip size for use in the HMA mixes and three slabs were prepared for each test to be carried out in the study. It should be noted that mixes 1 and 2B are the conventional mixes typically used in British Columbia. Alternatives were compared to both these mixes. However, the research to date only reports on the three tests carried out as discussed in the following paragraphs.

The focus of the study was on the rutting resistance and strength properties of the mixes containing both the recycling material quantities and rejuvenator. The study indicated that the conventional mixes (1 and 2B) exhibited the best rutting resistance properties and highest dynamic and resilient modulus values as well as mixes 4 and 6 demonstrated similar properties to conventional mixes. However, mixes with 3% RAS and 0.3% rejuvenator with or without RAP exhibited larger rutting depths and lower modulus values as shown in Figure 2-4 to Figure 2-6.

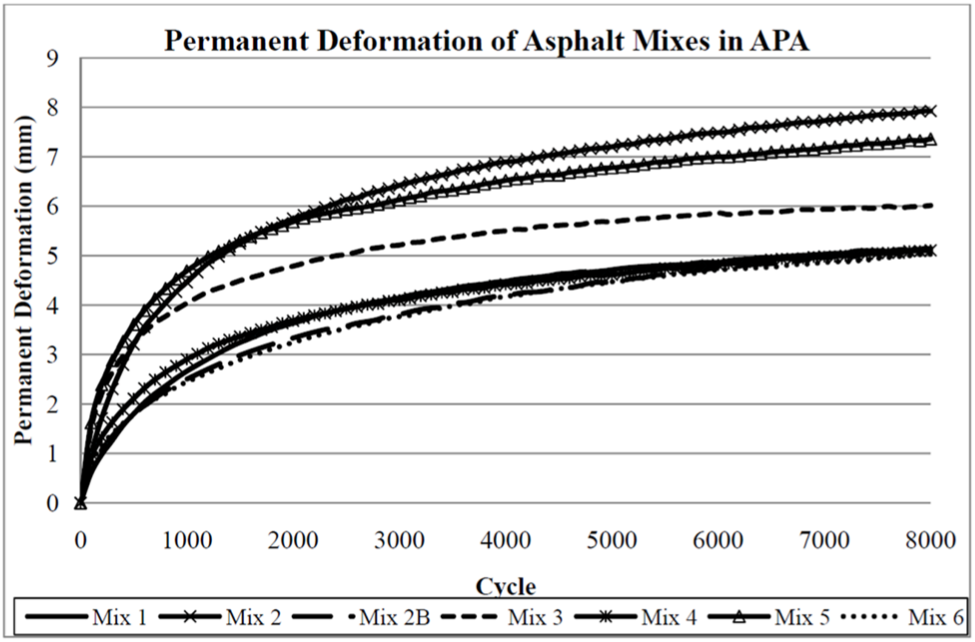


Figure 2-4: Average Permanent Deformation [Uzarowski 2010]

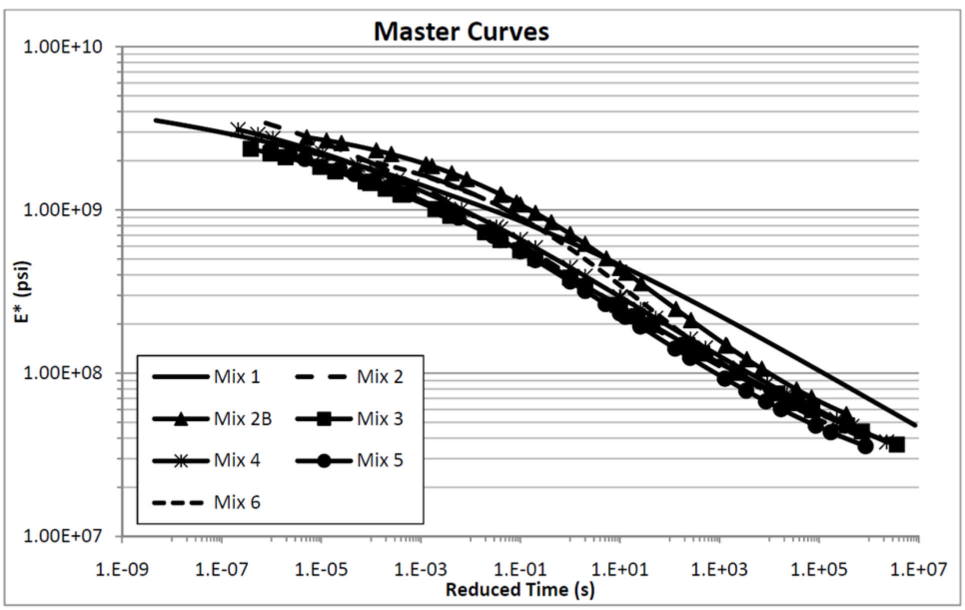


Figure 2-5: Dynamic Modulus master Curves on the Mixes [Uzarowski 2010]

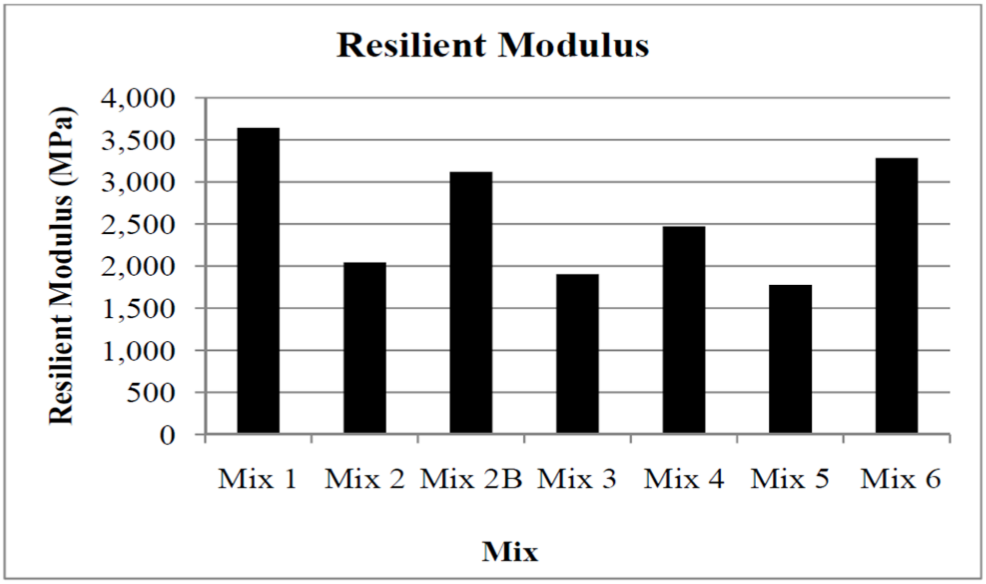


Figure 2-6: Average Resilient Modulus on all the mixes [Uzarowski 2010]

The research indicated a significant effect of the rejuvenator to the HMA mixes as shown in the results attained. It was observed that mixes with 3% RAS and rejuvenator reduced their rutting resistance and stiffness properties whereas increasing the RAS percentage to 5% seemed to improve the properties to be closer to conventional mix properties [Uzarowski 2010].

Therefore, it was concluded that RAS can perform well in typical Vancouver HMA pavements mix similar to conventional mixes for Vancouver conditions with basically no frost.

2.4.4 Minnesota Study

This research focused on analyzing the effects of recycled material such as new Manufactured Waste Scrap Shingle (MWSS) and demolished/Tear Off Scrap Shingle (TOSS) in HMA pavement mixtures. It addressed incorporating recycled material into pavement mixtures without compromising the environment and pavement durability. The study investigated the effects of RAS/RAP percentages, source of material, and asphalt binder grade on HMA properties to develop specification for the use of roofing shingles in HMA pavements. The study consisted of an extensive laboratory testing of laboratory produced RAS/RAP mixtures and field evaluations of in-place asphalt produced mixtures for the state of Minnesota. The source of the recycled materials did not have significant effect in performance. The mixes evaluated in study are given in Table 2-4 [McGraw 2010].

Table 2-4: Minnesota Pollution Control Agency Material Study Matrix [McGraw 2010]

Mix Description		Recycled Material			Binder	
Mix No.	**Mix ID**	**RAP (%)**	**TOSS (%)**	**MWSS (%)**	**PG 58-28**	**PG 51-34**
1	PG 58-28 Control	0	0	0	x	
2	15% RAP	15	0	0	x	
3	25% RAP	25	0	0	x	
4	30% RAP	30	0	0	x	
5	15% RAP, 5% MWSS	15	0	5	x	
6	15% RAP, 5% TOSS	15	5	0	x	
7	25% RAP, 5% TOSS	25	5	0	x	
8	25% RAP, 5% MWSS	25	0	5	x	
9	25% RAP, 5% TOSS (51-34)	25	5	0		x
10	25% RAP, 5% MWSS (51-34)	25	0	5		x
11	25% RAP, 3% TOSS	25	3	0	x	
12	25% RAP, 3% MWSS	25	0	3	x	
13	15% RAP, 3% TOSS	15	3	0	x	
14	15% RAP, 3% MWSS	15	0	3	x	
15	10% RAP, 5% TOSS	10	5	0	x	
16	15% RAP, 5% TOSS	15*	5	0	x	
17	5% TOSS	0	5	0	x	

* Difference in RAP source – milling containing 4% asphalt cement (AC)

The study demonstrated that RAP/RAS HMA mixtures have a strong correlation between the virgin asphalt binder content and the high/low performance grade temperature of the particular binder. The mixture dynamic modulus illustrated further correlation between dynamic modulus and the new binder content at higher temperature providing evidence of the relationship between virgin asphalt binder content and mixture durability. Dynamic modulus tests on the laboratory produced mixtures demonstrated significant differences in stiffness especially at lower frequencies (higher temperature) between the mixtures containing RAP/RAS and virgin mixtures. The Asphalt Pavement Analyzer (APA) was used to test for rutting susceptibility and the rut results showed a reduction in rut depth with increasing amounts of RAP/RAS content indicating increased stiffness in the mixture [McGraw 2010].

However, moisture sensitivity tests conducted on RAP/TOSS mixtures failed to meet the MnDOT specifications while RAP/MWSS mixtures showed higher values. Increased moisture sensitivity could mean potential decrease in durability. It was observed that thermal (low temperature) cracking heavily influenced durability of Minnesota HMA pavements. The low temperature performance grade (PG) was increased with addition of RAP/RAS suggesting an increase in thermal cracking potential of the mixture. However, this is an aspect still under investigation at the University of Minnesota [McGraw 2010].

The study further indicated that TOSS or demolished roofing shingles increased the mixture demand for new asphalt binder more than the new MWSS, which lowered the new binder to total binder ratio. Extraction process indicated that TOSS was stiffer than MWSS, which illustrated the increasing difference in mixtures containing TOSS or MWSS and RAP, as RAP content increased. However, the dynamic modulus demonstrated little difference between the two types of shingles at 3% level regardless of the RAP content. HMA mixtures containing TOSS exhibited more visual stiffness, the difference being more apparent at lower frequencies (higher temperature).

The study recommended the use of a softer asphalt binder (PG 51-34), as a harder asphalt binder (PG 58-28) had a dramatic effect on the properties of RAP/RAS mixtures. Dynamic modulus results showed reduced a stiffness and smother master curve which was further supported by field evaluations. [McGraw 2010].

Dynamic modulus master curves were used to compare laboratory-produced mixtures and asphalt plant-produced mixtures. The results demonstrated that greater mixing of recycled material and virgin asphalt binder in laboratory-produced mixes due to the longer period of heating yielding stiffer mixtures and improved resistance to rutting.

Six test sections were constructed using both MWSS and TOSS, the first four in 2005 and the last two in 2008. Pavement performance evaluation was carried out after 3 years to assess the pavement conditions, these roads included; Dakota County CSAH 26 (PG 58-34), US Highway 10 (PG 64-34), Hassan Township Park Drive (PG 58-28), Ramsey County Lower Afton Trail (PG 58-28), MnROAD Mainline (I-94), and Hennepin County CSAH 10. It was observed that the new asphalt binder to total asphalt binder ratio of the mixture was influenced by the amount and type of recycled material and using a softer binder grade demonstrated more performance benefits with less visual distresses confirming the previous laboratory analysis. The study showed little significant difference in field performance between new MWSS and demolished/TOSS, hence recommended further study in the usage of TOSS in HMA mixtures [McGraw 2010].

2.5 Laboratory Testing at CPATT

Six Ontario mixes were formulated by Miller Paving Ltd according to MTO specification. The mixes simulated two major construction pavement layers: surface layer and binder layer. The mixes were selected to represent a wide variety of applications from medium to low volume traffic loading; these mixes are given in Table 2-5. Mixes 1, 4, 5, and 6 are the surface layers whereas mixes 2 and 3 are the binder layers. Different performance grade asphalt binder (PG Grade) was used for different mixes according to the mix design. The control mix (HL 3 13.5% RAP and 1.4% RAS) was used to pave the CPATT Test Track and two residential streets in the Town of Markham. The control mix was obtained from Steed and Evans Ltd's Heidelberg plant while the other five mixes under study were prepared at CPATT laboratory with materials from Miller Paving Ltd [UL-Islam 2010].

Table 2-5: Description of the Design Mixes

Mix	Description of the Design Mix	PG Grade
Surface Layer		
Control Mix	Hot Laid 3 (HL 3), 13.5% RAP, 1.4% RAS	PG 58-28
4	Superpave 12.5 (SP12.5) FC1, 17% RAP, 3% RAS	PG 52-34
5	Superpave 12.5 (SP12.5) FC2, 6% RAS	PG 52-40
6	Superpave 12.5 (SP12.5) FC2, 12% RAP, 3% RAS	PG 52-34
Binder Layer		
2	Superpave 19 (SP19 E), 25% RAP, 3% RAS	PG 52-40
3	Superpave (19 SP19 E), 6% RAS	PG 52-34

2.5.1 Dynamic Modulus Test

Dynamic modulus, a fundamental parameter in pavement design, is a complex number defining the stress strain relationship of linear viscoelastic material under a continuous sinusoidal loading in other words it is the ratio of peak dynamic stress (σ_0) to the peak recoverable axial strain (ε_0) [Tashman 2007]. The phase angle is the angle which the axial strain lags behind the dynamic stress, and it characterises the viscous behaviour of the material for example the closer the phase angle to 90^0C, the more viscous the material. Dynamic testing was performed on six Hot Mix Asphalt mixes and the results are as shown in Figure 2-7. The dynamic modulus results attained in this study were analysed and used in the MEPDG performance analysis to evaluate the deterioration prediction models that incorporate d 1.5%, 3%, and 6% RAS in HMA pavement mixtures for this research.

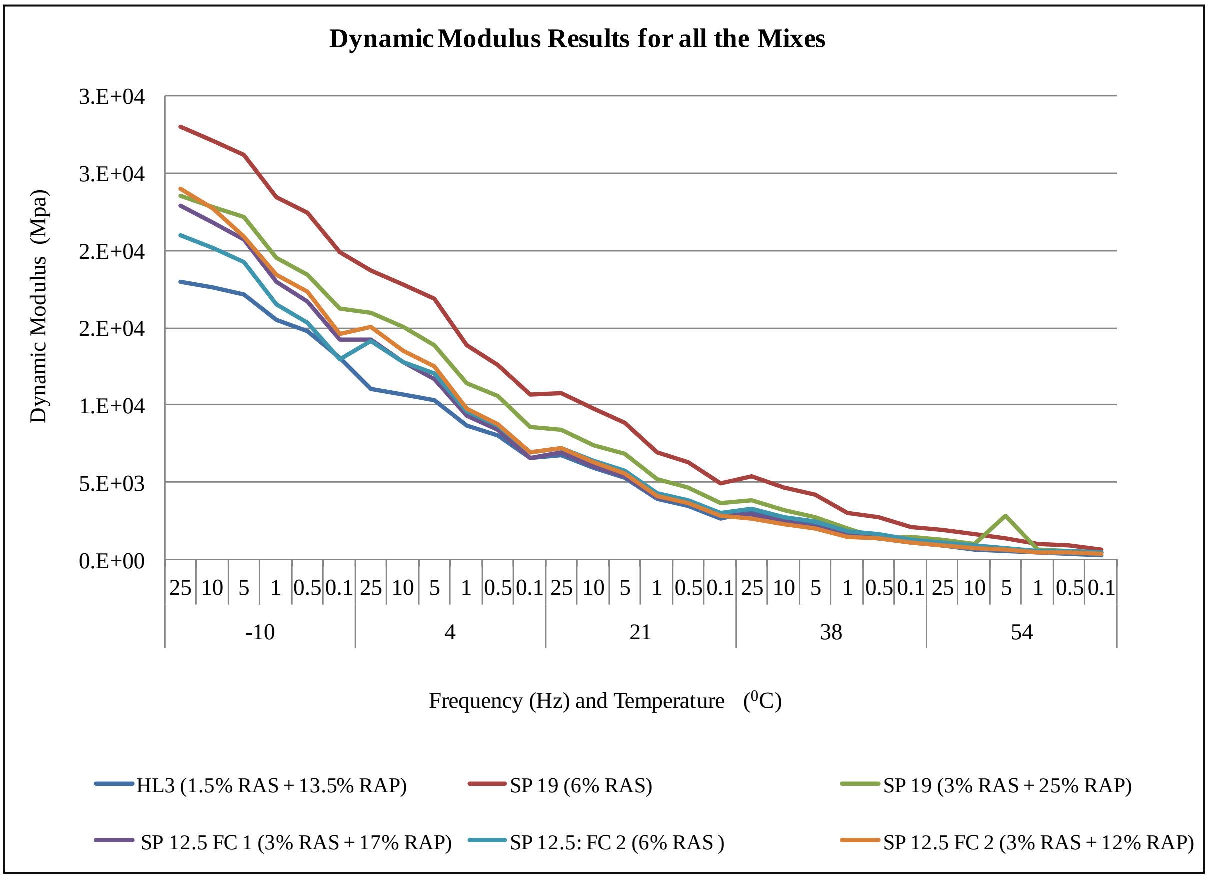


Figure 2-7: Dynamic Modulus Results [UL-Islam 2010]

To reduce rutting potential, a high dynamic modulus at high temperatures is desirable, while low Dynamic Modulus at low temperatures reduces fatigue cracking potential. From the study, at low temperatures mixes SP12.5 FC2 containing 3% RAS and 12 % RAP (a surface layer mix) and SP 19E containing 6% RAS (a binder layer mix), had the highest dynamic modulus than other mixes, indicating higher fatigue cracking susceptibility.

At high temperatures, mixes HL 3 containing 1.5 % RAS and 13.5 % RAP (a surface layer mix) and SP19 E containing 3% RAS and 25 % RAP (a binder layer mix) had the lowest dynamic modulus than other mixes, indicating lower resistance to rutting [UL-Islam 2010].

2.5.2 Resilient Modulus (M_r) Test

Resilient modulus is a fundamental material property that characterizes unbound pavement material and a required input parameter in the mechanistic-empirical pavement design method. The resilient modulus measures the material stiffness and provides a way of analyzing the stiffness of materials under different conditions, such as moisture, density and stress level. Resilient modulus is typically determined through laboratory tests by measuring stiffness of a cylinder slab subject to a cyclic axle load. The data attained can be used as an input for pavement design, evaluation, and analysis. Resilient Modulus was performed on all the hot asphalt mixes incorporated with RAS and compared with preceding studies.

The study showed that for the surface layer mixes, HL 3 (1.5% RAS and 13.5 % RAP) had the highest total and instantaneous resilient modulus indicating increased potential for thermal cracking whereas SP 12.5 FC2 (6% RAS) had the lowest resilient modulus among the surface mixes. For the binder layer mixes, SP 19 (6% RAS), had the higher resilient modulus compared to SP 19 (3%RAS and 25% RAP) [UL-Islam 2010]. Table 2-6 gives the results attained for the design mixes while Figure 2-8 demonstrates the mixes with more resistance to cracking.

Table 2-6: Resilient Modulus Results (MPa) [UL-Islam 2010]

Mix	Average Total Resilient Modulus (Mpa)	Average Instantaneous Resilient Modulus (Mpa)	Total Poisson Ratio	Instantaneous Poisson Ratio
HL 3 (1.5% RAS and 13.5% RAP)	3108	2728	0.28	0.3
SP12.5 FC1 (3% RAS and 17% RAP)	1318	1318	0.32	0.32
SP12.5 FC2 (3% RAS and 12% RAP)	1159	1154	0.34	0.34
SP12.5 FC2 (6% RAS)	1008	1016	0.34	0.34
SP19 (3% RAS and 25% RAP)	1416	1411	0.29	0.25
SP19 (6% RAS)	1682	1714	0.25	0.3

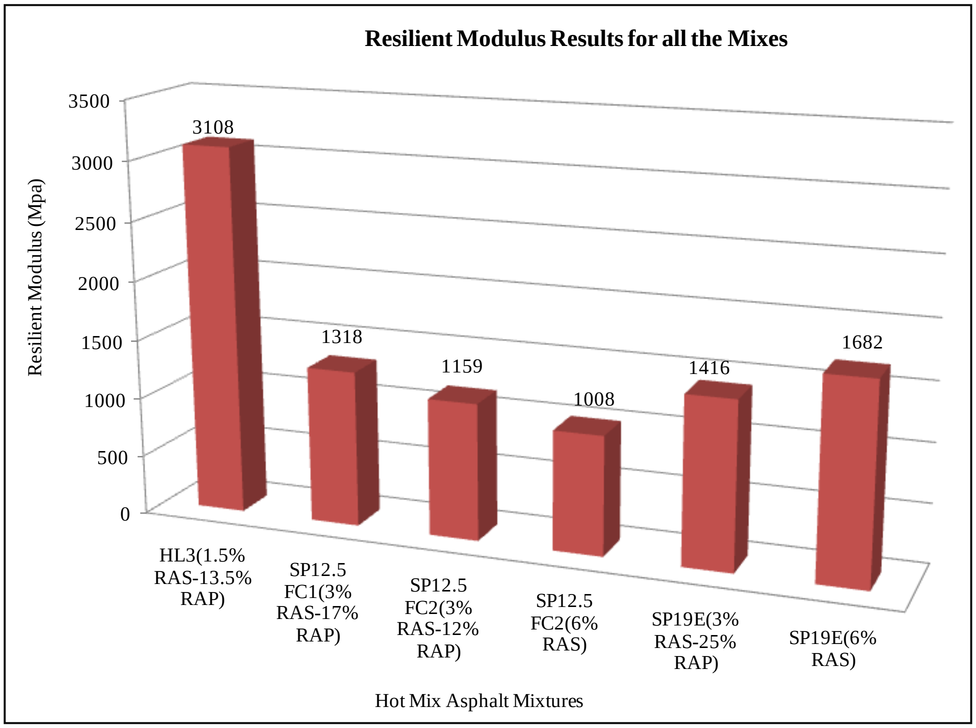


Figure 2-8: Resilient Modulus Results for all mixes [UL-Islam 2010]

2.5.3 Thermal Stress Restrained Slab Testing (TSRST)

Thermal Stress Restrained Slab Test was carried out to determine and simulate the tensile strength and temperature at fracture of laboratory compacted bituminous mixtures through measuring the tensile load in the slab which cooled at a constant rate while being restrained and contracted. It was observed that failure temperature was lowest for mixes SP12.5 FC2 6% RAS and SP19 3% RAS and 25% RAP indicating an increase in thermal cracking resistance as shown in Figure 2-9. Mixes with a lower RAS percentage simulated similar performance traits as conventional mixes [UL-Islam 2010]. Table 2-7 summarizes the TSRST results attained.

Table 2-7: Thermal Stress Restrained Slab Testing (TSRST) Result [UL-Islam 2010]

Mix Description	Slab	Stress (MPa)	Mean Stress (MPa)	Std Dev Stress (MPa)	Temp (°C)	Mean Temp (°C)	Std Dev Temp (°C)
HL 3 (1.5% RAS, 13.5 %RAP)	1	1.4	1.2	0.3	-22	-19	3.8
	2	0.9			-16		
SP12.5 FC1 (3 % RAS, 17 % RAP)	1	2.2	1.9	0.4	-30	-29	0.8
	2	1.7			-28		
SP12.5 FC2 (3 % RAS, 12 % RAP)	1	3.2	2.7	0.7	-30	-32	3.3
	2	2.3			-34		
SP12.5 FC2 (6 % RAS)	1	2.1	2.0	0.1	-36	-36	0.8
	2	2.0			-37		
SP19E (3 % RAS, 25 % RAP)	1	2.1	2.2	0.2	-33	-35	3.0
	2	2.3			-37		
SP19E (6 % RAS)	1	1.4	1.3	0.1	-30	-26	5.2
	2	1.2			-22		

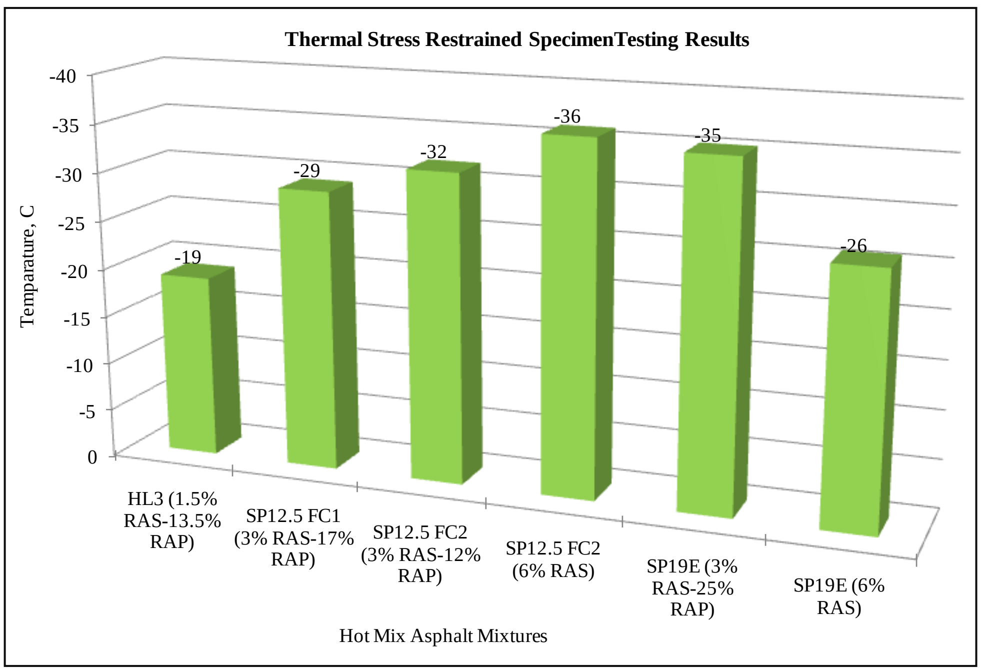


Figure 2-9: Thermal Stress Restrained Slab Testing Result [UL-Islam 2010]

2.5.4 Flexural Bending Beam Test

The Flexural Bending Beam Test measures fatigue resistance which is the capability of the material to withstand repeated bending without failure. There is a correlation between the measured repeated deflection and fatigue of asphalt pavement [Hveem 1955]. The fatigue resistance test was carried out on all the design mixes; Mixes SP12.5 FC1 (3% RAS, 17% RAP) and SP12.5 FC2 (6% RAS) had the highest resistance to fatigue failure whereas HL 3 had the lowest susceptibility to failure given in Table 2-8 and shown in Figure 2-10 [UL-Islam 2010]. The author suggested that more research basing on the tests be carried out because the base layers mixes illustrated unexpected results.

Table 2-8: Flexural Fatigue Test Results [UL-Islam 2010]

Mix	Air Voids (%)	Failure Point (Cycles)	Mean (Cycles)	Std Deviation
HL 3 (1.5% RAS, 13.5% RAP	7.8	24,399	23,899	707
	7.1	23,399		
SP12.5 FC1 (3% RAS, 17% RAP)	7.9	88,199	79,849	11,809
	7.1	71,499		
SP12.5 FC2 (3% RAS, 12% RAP)	8.2	38,999	37,549	10,865
	7.6	36,099		
SP12.5 FC2 (6% RAS)	6.9	70,699	70,299	2,051
	7.2	69,899		
SP19E (3% RAS, 25% RAP)	10.2	19,999	20,999	141
	10	20,199		
SP19E (6% RAS)	9.8	7,199	9,899	3,818
	10.6	12,599		

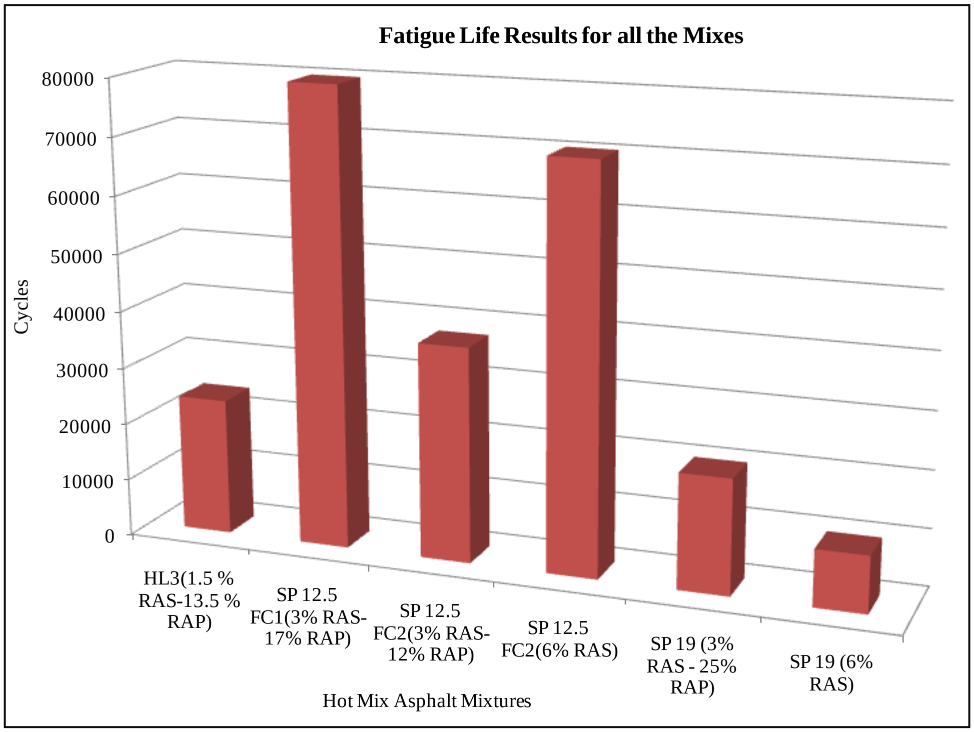


Figure 2-10: Fatigue Life for all the Mixes [UL-Islam 2010]

2.6 Pavement Performance Evaluation

Pavement performance is the measure of the in-service pavement conditions expressed in terms of the pavement condition index such as structural performance (distress manifestations), and functional performance (serviceability). Pavement prediction models can be grouped into three general types [NordFoU 2006]:

- Empirical Based – this depends on certain measured/estimated variables like deflection, accumulated traffic loads related to serviceability loss or measures of deterioration versus pavement age through regression analysis.
- Mechanistic-Empirical Based- this depends on certain calculated responses like subgrade strains, pavement layer stresses/strains combined with accumulated traffic loads related to serviceability loss or measures of deterioration versus age through regression analysis (coefficients are determined)

- Experience Based – this depends on experience where serviceability loss or deterioration measures versus age are estimated for a combination of variables using Markovian transition models or Bayesian model.

For the purpose of this research, the Mechanistic-Empirical method and experience based method were used in the analysis and development of LCCA models based on the deterioration rate as well as field performance simulations. Figure 2-11 illustrates the factors that affect pavement performance over the service life.

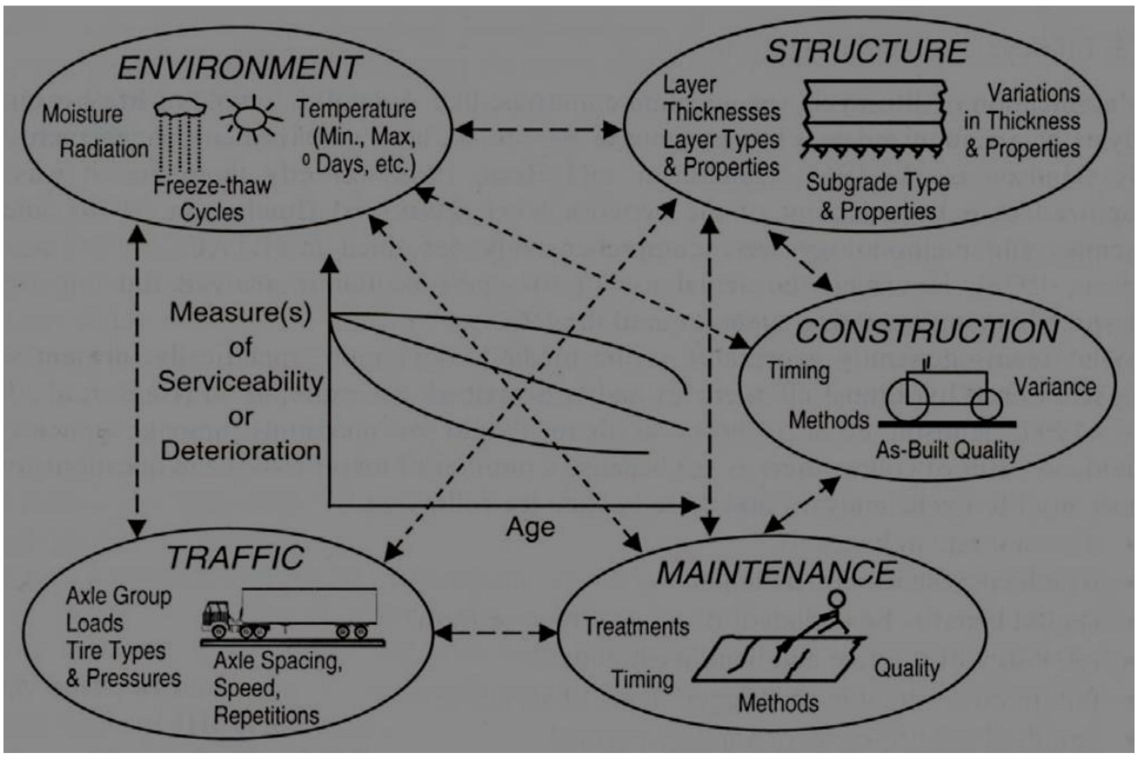


Figure 2-11: Factors affecting Pavement Performance [Tighe 2007]

2.6.1 Pavement Distress Evaluation

Surface distress analysis evaluates the types of pavement distresses, density, and severity observed on the pavement surface. Overtime, as expected, the distresses tend to develop due to traffic loading, pavement surface age, and pavement condition. The types of pavement surface distresses can be categorized as [MTO 1989]:

- Surface Defects such as ravelling and course aggregate loss, and flushing.

- Surface Deformations such as ripping and shoving, wheel track rutting, and distortion.
- Cracking such as longitudinal wheel track, centreline, pavement edge, transverse, longitudinal meander and mid-lane, and random. The cracking can be further identified as either single or multiple cracks/alligator cracks.

The severity of the pavement distresses were categorized as: very slight, slight, moderate, severe, and very severe [MTO 1989].

2.7 Pavement Performance Modeling

2.7.1 Mechanical Empirical Pavement Design Guide (MEPDG)

Pavement design was previously based on experience only in North American and Canadian agencies. The future direction in pavement design is to use the mechanistic-empirical approach [Dzotepe 2010]. The calculated pavement responses such as stresses, strains and deformations (mechanistic) are adjusted accordingly based on empirical performance models.

In the mid-1990s, AASHTO embarked on research for a new pavement design guide and identified some critical features such as mechanistic, empirically calibrated, allowing for user calibration, including existing theory and models, and creating software which provided a rational engineering approach. This became the mechanistic-empirical approach to pavement design known as U.S. National Cooperative Highway Research Program - NCHRP 1-37A Project [AASHTO]. The software interface is as shown in Figure 2-12.

Mechanistic-Empirical design focuses on pavement performance taking into account factors that effect its performance such as materials, climate, traffic loads, and construction procedures to estimate the pavement distress condition over the design period. The software involves three levels of performance analysis types, namely:

- Level 1 – represents the most accurate level usually used on major highways, interstate, and strategic roads or where specific material characterization is required. This involves both field and laboratory testing of the material to be used in the pavement layers (surface layer, and granular base/subbase) and subgrade soil.
- Level 2 – represents the medium accuracy input level used when there is limited testing of the material used in pavement layers. Level 2 assumes the mechanical, physical or chemical

properties based on previous experience in case of no laboratory or field testing performed on the material to be used.

- Level 3 – represents the least accuracy level usually used for low volume roads as well as when laboratory or field testing for the materials is unavailable. Default values are recommended by the local agencies for material characterization used in this input level.

The pavement deterioration can be expressed in months or years hence serviceability loss over time can be analysed.

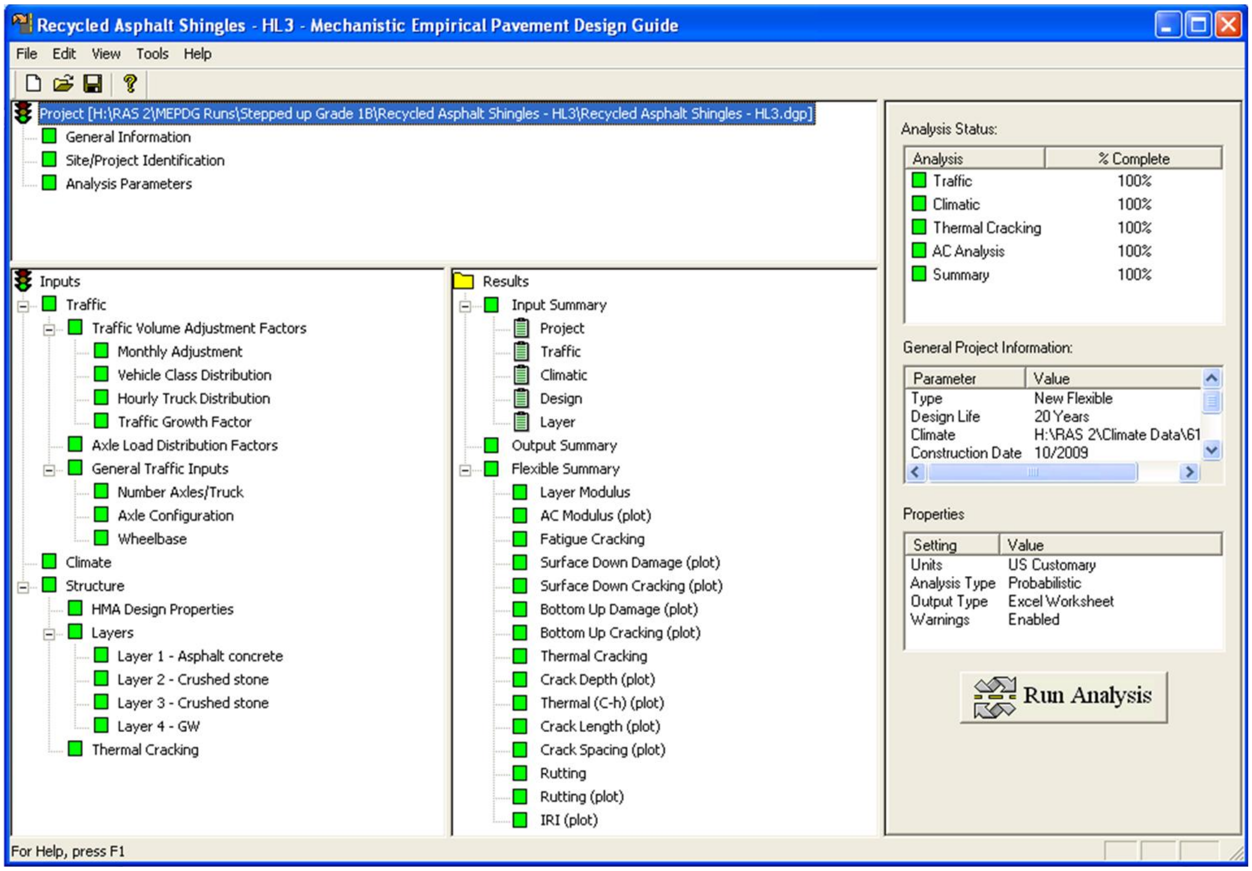


Figure 2-12: Main MEPDG Interface Illustrating the Components Involved

2.7.1.1 ***Pavement Response Models***

The MEPDG produces two types of pavement response models namely: (1) an environmental effects model that stimulates the time-and-depth dependent temperature and moisture conditions in the pavement structure in response to climatic conditions and (2) a structural response mode that determines stresses and strains at critical locations within the pavement structural system in response to traffic loading.

Environmental Effects

Environmental factors tend to induce pavement responses and distresses over the analysis design period. The seasonal climatic fluctuations (moisture and temperature) are induced by changes in ground water table, precipitation/infiltrations, freeze-thaw cycles, and any other external factors that are incorporated within the MEPDG software via the Enhanced Integrated Climatic Model (EICM). EICM program simulates the behavioural, pavement characteristics, and subgrade changes that are induced by the environmental factors. With the flexible pavement, EICM evaluates the following environmental effects [Schwartz 2007]:

- Seasonal changes in moisture content for unbound materials and subgrade.
- Changes in resilient modulus M_R for all unbound materials and subgrade caused by changes in soil moisture content and freeze thaw cycles.
- Temperature distribution in bound asphalt cement layers which determines temperature-dependent asphalt cement properties

The environmental factor incorporated in the MEPDG is a coefficient that is multiplied by the resilient modulus at the optimum moisture and density condition to obtain the seasonally adjusted resilient modulus as a function of time and depth.

Structural Response

The pavement system uses the mechanistic structural response models to determine the stresses, strains, and displacements caused by traffic loading and influential environmental conditions. The MEPDG uses two lines of theory in determining structural responses, namely;

a) For materials assumed to be linear-elastic, a Multilayer Elastic Theory (MLET) is used to determine the pavement responses.
b) For unbound materials, a non-linear Finite Element (FE) code is used to determine the pavement stresses, strains and displacements.

The MEPDG provides performance predictions by identifying locations in the pavement structure where critical pavement responses (stresses and strains) will attain their most extreme values. An

MEPDG analysis evaluates critical responses at several depth locations in the pavement structure depending on the distress type [Schwartz 2007]:

- Fatigue (pavement structure Z=0, 0.5 inches from the surface Z=0.5, and bottom of each bound/stabilized layer)
- Rutting depth (mid-depth of each layer/sub layer, top of subgrade, 6 inches below the top of subgrade)

2.7.1.2 ***Pavement Performance Prediction Models***

MEPDG evaluates pavement performance in terms of individual distress models or empirical distress models using transfer functions that are incorporated into the software to determine the major structural distresses. The models also estimate pavement smoothness as a function of individual structural distresses and any other factors.

Damage Vs. Distress

Seasonal calculations are used to determine distresses such as rutting in the asphalt layers in the flexible pavement. The empirical model is expressed as shown in Equation 2-1.

$$\frac{\varepsilon_p}{\varepsilon_p} = \beta_{r1} a_i T^{a_2 \beta_{r2}} N^{a_3 \beta_{r3}}$$

Equation 2-1

Where; ε_p is accumulated plastic strain for N repetitions of load

ε_r is resilient strain of asphalt material as a function of mix properties, temperature and the rate of loading

N is number of load repetitions

a_i is the regression coefficient

β_{r1} is the field calibration coefficients

Equation 1 is evaluated at the mid-thickness of each sub layer (asphalt layers are divided into sub layers). The total rutting R_d from the sub layers is the sum of the total rutting ΔR_{di} from each sub layer i given in Equation 2-2 and Equation 2-3.

$$\Delta R_{di} = \varepsilon_{pi}\,\Delta h_i$$

Equation 2-2

$$R_d = \sum_{i=1}^{l} \Delta R_{di}$$

Equation 2-3

The distresses that cannot be directly evaluated are quantified in terms of damage factors such as the alligator fatigue cracking empirical model shown in Equation 2-4.

$$N_f = \beta_{f1} K_{1_t} (\varepsilon_t)^{\beta_{f2} K_2} (E)^{\beta_{f3} K_3}$$

Equation 2-4

Where; N_f is the number of repetitions to fatigue cracking failure

ε_t is the tensile strain at the critical location

E is the asphalt cement stiffness at appropriate temperature

K_1, K_2, K_3 are the regression coefficients from laboratory fatigue tests

$\beta_{f1,}$ $\beta_{f2,}$ β_{f3} are the field calibration coefficients

Accumulated fatigue damage is based on Miner's Law, Equation 2-5;

$$D = \sum_{i=1}^{T} \frac{n_i}{N_{fi}}$$

Equation 2-5

Where; D is the damage

T is the total number of seasonal periods

n_i is the actual traffic for period i

N_{fi} is the traffic repetitions causing the fatigue failure under prevailing conditions for the period i

The determined damage factor is then related to observed fatigue distress qualities during the field calibration process.

Distress Models

The empirical distress prediction models in MEPDG explain the following structural distresses as:

a) Permanent Deformation (Rutting)

   It is observed within the asphalt cement layer, unbound base or subbase layer and subgrade layer.

b) Fatigue Cracking

   This kind of distress can be observed within the asphalt cement layer such as bottom-up (alligator cracking) and top-down (longitudinal fatigue cracking) or within the cement stabilized layer.

c) Thermal cracking

Pavement Smoothness

Pavement smoothness, often used as a complex index of pavement quality, is influenced by the distress modes in the flexible pavement systems. Smoothness is directly related to overall ride quality, a factor which is very important for pavement users therefore empirical smoothness prediction models are incorporated within the MEPDG for performance analysis [Schwartz 2007].

The MEPDG characterizes the pavement smoothness in terms of International Roughness Index (IRI). It provides IRI prediction models as a function of pavement type, base type, and construction type.

Design Reliability

Due to the large amount of uncertainty and variability in pavement design and construction, traffic loading, and climatic factors over the analysis period, MEPDG software largely focuses on design reliability in prediction of distresses. MEPDG has reliability levels and standard deviations for each distress model predicted by the mechanistic empirical computations. Future maintenance cost is or should be lower for the higher reliability design [Schwartz 2007].

Pavement Performance criteria

A performance criterion defines the maximum amounts of individual distresses or smoothness acceptable to an agency at a given level of reliability. These are user friendly inputs in the design process.

2.7.2 Life Cycle Cost Assessment (LCCA)

LCCA is an analytical technique that uses economic principles to evaluate long-term alternative investment options by analyzing total cost comparisons of competing design alternatives with equivalent benefits [Caltrans 2010].

The analysis takes into account the relevant costs of the agency, owner, facility operator, and the pavement users that will occur throughout the design life of the alternatives. The relevant costs include initial construction and project support, future maintenance and rehabilitation, and the user costs (delays and vehicle costs). The best appropriate time to perform LCCA for a particular project is when it is in its design stages as it helps in decision making as shown in Figure 2-13 and Figure 2-14.

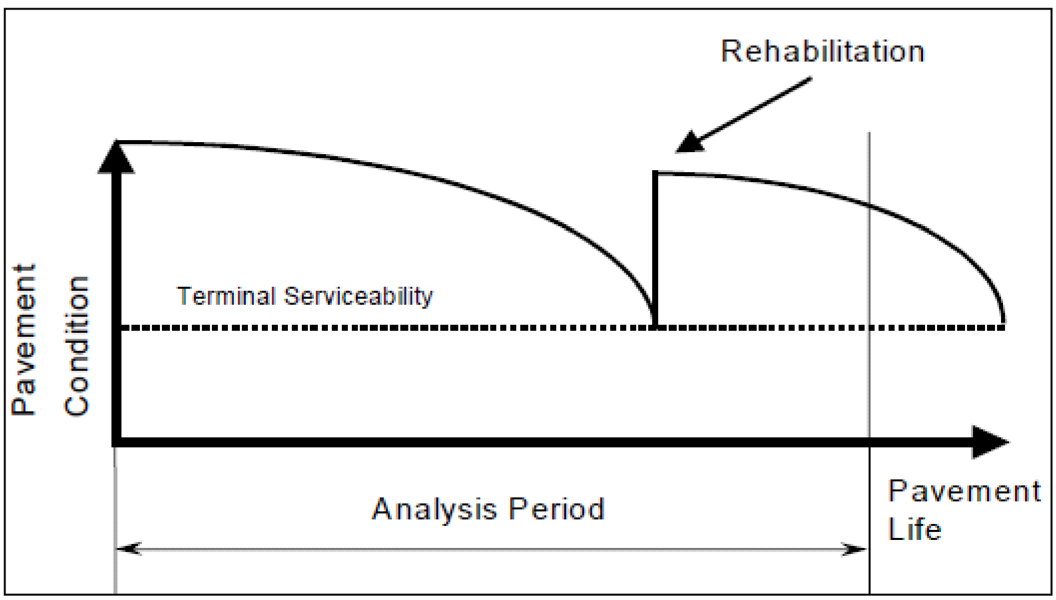


Figure 2-13: Pavement Analysis for a strategy [Caltrans 2010]

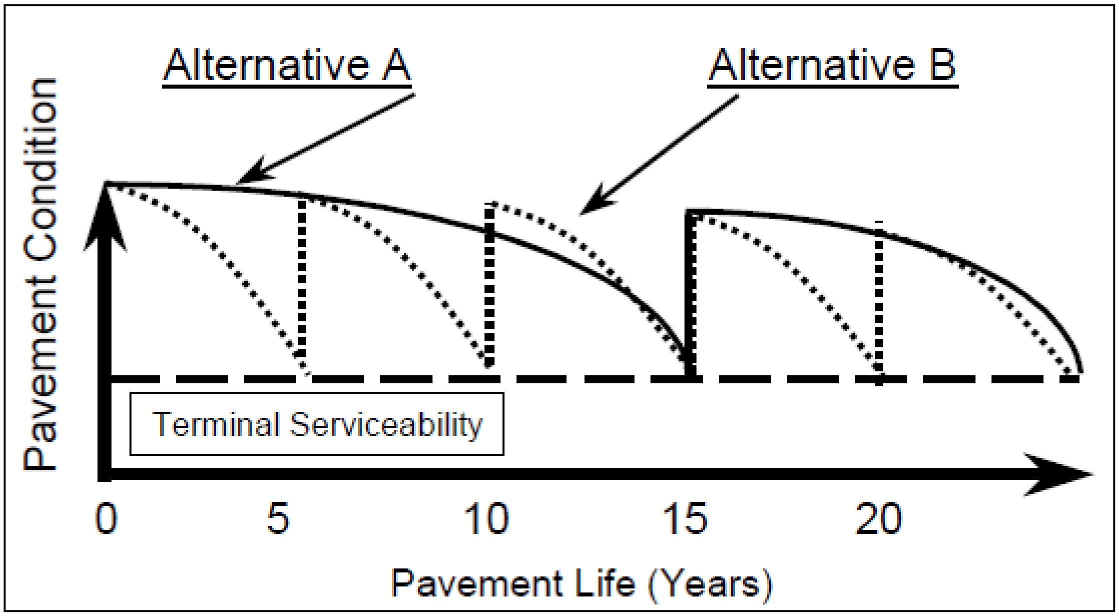


Figure 2-14: Performance Curve for two Strategies [Caltrans 2010]

Historically, public and private agencies in the United States and Canada have recognized LCCA as an effective tool to assist in the selection of pavement design alternatives. In 1986, AASHTO encouraged the use of LCCA in its design guide for pavement structures to evaluate the cost effectiveness of the alternative designs [AASHTO 1986]. For mixtures with recycled materials such as recycled asphalt shingles, asphalt rubber, and recycled pavement aggregates to be widely accepted in pavement design, they must be shown to be cost effective for example lower life cycle cost than the alternatives [Hicks 1999].

The user conducting LCCA needs to be aware of the inherent uncertainty surrounding the variables used as inputs into the analysis such as from assumptions, estimates, and projects [Walls III 1998]. A summary of LCCA input variables and the general basis used to determine their values is given in Table 2-9.

Table 2-9: LCCA Input Variables [Wall III 1998]

LCCA Component	Input Variable	Source
Initial and Future Agency Costs	Preliminary Engineering	Estimate
	Construction Management	Estimate
	Construction	Estimate
	Maintenance	Assumption
Timing of Costs	Pavement Performance	Projection
User Costs	Current Traffic	Estimate
	Future Traffic	Projection
	Hourly Demand	Estimate
	Vehicle Distributions	Estimate
	Dollar Value of Delay Time	Assumption
	Work Zone Configuration	Assumption
	Work Zone Hours of Operation	Assumption
	Work Zone Duration	Assumption
	Work Zone Activity Years	Projection
	Crash Rates	Estimate
	Crash Cost Rates	Assumption
NPV	Discount Rate	Assumption

2.7.3 Environmental Analysis using PaLATE

Pavement Life Cycle Assessment Tool for Environment and Economic Effect (PaLATE) is an Excel-based life-cycle analysis tool that analyses environmental and economic information to evaluate the use of different materials both virgin and recycled materials in the construction and maintenance of pavements. It calculates the cumulative environmental effects over the design analysis period such as energy consumption and air pollution as well as the economic net present value and annualized cost of two concurrent options. “The computer-based decision support tool integrates economic analysis and environmental assessment of the pavements” [Horvath 2003].

PaLATE software assesses the emissions associated with the materials at production, construction, transportation, and maintenance of asphalt pavements, subgrade, embankment, and shoulder materials incorporating both virgin and recycled materials. The energy consumption emissions such as CO_2, NO_x, PM_{10}, SO_2, CO, as well as determines the average leachate releases for the different construction materials, water consumption, mercury, and lead [Horvath 2003].

PaLATE software can also calculates the net present value (NPV) of the pavement over its life cycle and annualized pavement costs as well as allows for sensitivity analysis using the different construction and maintenance schedules, and two different discount rates.

2.8 Summary

Based on the literature review several gaps have been identified in the usage of RAS in HMA. Currently, limited data is available on demolished or post-consumer or tear-off RAS and particularly for usage in Canada. The gaps identified include;

- Evaluate the performance of pavement constructed with HMA containing RAS (post-consumer shingles) and/or RAP.
- Predict the pavement service life of post-consumer RAS pavements using modeling design guides such as MEPDG
- Perform a life-cycle assessment of post-consumer RAS pavements using PaLATE and LCCA
- Perform a benefit-cost analysis (BCA) to evaluate if the savings are worthwhile by using post-consumer RAS in pavements.

Chapter 3: Research Methodology

3.1 Introduction

The University of Waterloo's Centre for Pavement and Transportation Technology (CPATT), Canada has had ongoing research since 2007 to evaluate the optimal demolished RAS usage in various Ontario HMA pavements. The studies have shown promise in the use of 1.5% RAS with 13.5% RAP in asphalt pavements. The present research study involved the need to quantify and qualify the maximum or optimal percentage of RAS in HMA pavement mixtures without compromising on the pavement performance, economic and environmental aspects. All the RAS used in the study were demolished roofing shingles.

The purpose of this chapter is to present the research methodology. The scope of the study consisted of three main parts: field and laboratory pavement condition evaluations, pavement structural analysis and deterioration prediction analysis using MEPDG, and life cycle assessment of the pavement (PaLATE and LCCA). Guidelines were developed for the usage of RAS in HMA. For the purpose of this thesis, six mix designs were evaluated and designed in partnership with the industrial partner, Miller Paving Limited given in Appendix A.

3.2 Source of Data

The data (dynamic modulus for pavement performance and asphalt binder specification for mix proportioning) used in the study was obtained from Phase two testing carried out at CPATT laboratory, pavement structural design in accordance to the Ontario Provincial Standard Specification (OPSS) from Ministry of Transportation Ontario (MTO), laboratory testing, and field evaluation of the four test sections located in the town of Markham and at the CPATT Test Track. The general information such as pavement life cycle unit costs per km, design life, initial International Roughness Index (IRI), traffic volume and truck percentages, traffic growth, mileage, pavement layer and pavement type were determined using the MTO pavement design specifications.

The pavement performance analysis data consisted of: (1) climate data (Toronto climate data file) was used in the MEPDG. For further analysis, a climate data file was downloaded from Environment Canada for Ontario to estimate the number of freeze-thaw cycles per year. (2) Pavement structural design data which comprised of asphalt cement layer (binder grade, volumetric, and gradation), unbound base/subbase, and subgrade materials (resilient modulus, and material classification) for parametric sensitivity analysis. Level 1 was used for the asphalt layer while for the unbound base, subbase, and subgrade materials, Level 3 default values were considered for all the material and design inputs except for gradation. Gradation of the material was done according to OPSS. The binder grade (viscosity), mixture volumetric (air voids, and effective binder content), and mix type (maximum nominal size and gradation) were the most important inputs in the MEPDG. These properties represent the design inputs to the empirical model for the dynamic modulus $|E^*|$, that is primarily the asphalt material property in the MEPDG analyses. The MEPDG was performed based on the dynamic modulus test results obtained in the CPATT laboratory and the asphalt binder properties from the asphalt supplier, McAsphalt Industries Limited to analyze the performance of each mix over the design life of the pavement. (3) Traffic inputs were obtained from a Ministry of Transportation Ontario design guide report [ARA 2011]. The CPATT Test Track pavement structure was used in this pavement performance study along with the MTO specifications and Transportation Association of Canada Pavement Design Guide [TAC 1997].

Life-Cycle Assessment involved two aspects (1) the quantification of sustainability through PaLATE to calculate the expected energy for using RAP and/or RAS in HMA pavements and greenhouse gases (GHG) emitted into the environment. The CPATT Test Track pavement structure was used for this environmental analysis. (2) The quantification of the economic benefits involved using the LCCA. The unit costs were obtained from the MTO LCCA reports and used to determine the overall initial costs for mixes when used in HMA pavements.

Table 3-1: Research Methodology Summary

Description	Activity and Testing	Remarks
Laboratory Testing: Asphalt Slabs		
Preparation and Construction of Asphalt Slabs	Superpave Gyratory Compactor	• Three slabs per mix (18 slabs)
Determine percent air voids	Maximum Relative Density (ASTM D 2041-03)	• 7 ± 1 had to be achieved
	Bulk Relative Density (AASHTO D: T166-07)	
Surface Characteristics	Surface Distress Evaluation – Visual Analysis	• Physical Properties of Asphalt slabs were measured such as mass, height and diameter • Initial, and end of first year and second year Freeze-thaw cycles • Statistical Analysis (F-Test, T-Test and ANOVA)
	Surface Texture – Sand Patch Method (ASTM E 965-96)	
	Friction – British Pendulum Tester (ASTM E 303-93)	
Field Pavement Evaluations		
Town of Markham	Surface Distress Survey – Visual Analysis	• Three residential streets of low traffic loading were evaluated
CPATT Test Track	Surface Distress Evaluation – Visual Analysis	• Comparison with previous evaluation • Statistical Analysis (F-Test, T-Test and ANOVA)
	Deflection Measurement - PFWD	
	Friction – British Pendulum Tester (ASTM E 303-93)	
Computer-Based Modeling		
Structural Analysis	MEPDG (Dynamic Modulus and Binder properties)	• Performance prediction modeling • Statistical Analysis (F-Test, T-Test and ANOVA)
Life-Cycle Assessment (LCA)	Environmental Impact Assessment - PaLATE	• Comparison between the design mixes
	Economic Assessment - LCCA	

3.2.1 Experimental Sites

In order to validate the performance model for the specific use of RAP and/or RAS in HMA pavements, field data was collected on the sections that are representative of design, environment and traffic conditions observed on the pavements. Therefore, four test sites under the study; one is located in the Regional Municipality of Waterloo (heavy traffic loading) while the other three test sections are located in the Town of Markham (low traffic residential streets). Figure 3-1shows the project locations of the RAS incorporated pavement in the Regional Municipality of Waterloo Waste Management Facility, which was paved in October 2009.

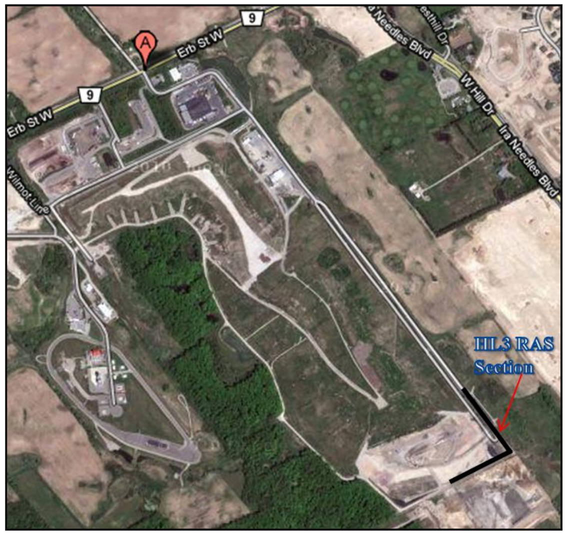


Figure 3-1: Satellite View from Google Maps of the RAS Section at CPATT Test Track

Figure 3-2 shows the residential streets which were paved in 2007 with RAS in the Town of Markham by Miller Paving Limited. This was to assess the performance of the overlays which incorporated RAS in their mixes. The three streets were located in low traffic volume areas and designed without parking space or walkways [UL-Islam 2010].

Figure 3-2: Satellite View of RAS Residential Streets in Town of Markham [UL-Islam, 2010]

3.3 Recycled Asphalt Shingles (RAS) Slabs

3.3.1 Construction of the Recycled Asphalt Shingles Slabs

Asphalt slabs were prepared and constructed at the Centre for Pavement Technologies and Transportation (CPATT) laboratory using the Rainhart Superpave Gyratory Compactor (SGC) for compaction. A set of three Slabs per mix were made and all material properties evaluated. The mixes were already made prior to the slab construction and stored in cardboard boxes. The mixes were then heated up for 20minutes, mixed to prevent segregation and measured into 3kg portions onto the laboratory trays. They were then transferred to the oven and heated up to 140^{0}C as specified by the mix design. The heated mix was then placed into the gyrator mould and compacted to 131^{0}C as shown in Figure 3-3.

A total of eighteen slabs in form of cylindrical prism were made, three slabs per mix as shown in Figure 3-4. Table 3-1 gives the number of gyrations and air voids attained per slab in the laboratory.

Figure 3-3: Rainhart Superpave Gyratory Compactor

Figure 3-4: Compacted Asphalt Slabs

3.3.2 Evaluation of Percent Air Voids

The Bulk Relative Density (BRD) test was carried out on each slab to determine the number of air voids in the slab [AASHTO 2007]. The Maximum Relative Density (MRD) per mix was carried out for the RAS Mixes according to ASTM D 2041-03 [ASTM 2003]. Figure 3-5 shows the bulk relative density carried out in the laboratory on the slab.

The samples were cleaned with a brush to remove any loose materials. They were then weighed and the dry mass was recorded. The asphalt slab was then submerged in a water bath (at 25 ± 1^{0}C) for 4±1 minutes and its mass in water was recorded. It was then removed, quickly surface dried of excess water with a damp towel and reweighed. In accordance with the specification the Bulk Relative Density (BRD) was then calculated using Equation 3-1 while the air voids were calculated from the MRD for each mix.

$$BRD = \left({}^{A}\!/_{(B - C)} \right)$$

Equation 3-1

Where;	A	-	Mass of dry slab in air, g
	B	-	Mass of saturated slab in air, g
	C	-	Mass of slab submerged in water, g

The air voids were calculated from Equation 3-2.

$$Percent\ Air\ Void = \left({(MRD - BRD)}/{MRD} \right) * 100$$

Equation 3-2

Figure 3-5: Bulk Relative Density Test carried out on the Slab

The temperature and number of gyrations per mix were kept constant among constructed samples (A – C). The differences in the percent air voids achieved per slab were as given in Table 3-2. Slight variations between samples are expected based on gradations, percentage of RAS and RAP. It was a challenge achieving 7% ± 1% within the same mix. Across the mixes, the percent air voids were 6.3% to 8.6%, resulting in an average of 7.30%. As long as the percent air voids between samples was in the 2% range, the slabs were subjected to freeze-thaw cycling.

Table 3-2: Number of Gyrations and the attained air void per Slab

Description of Mix	Number of Gyrations	Percent Air Voids				
		Slab A	Slab B	Slab C	Average	Standard Deviation
Mix 1: HL 3 1.5% RAS and 13.5% RAP	20	7.2	7.2	7.4	7.3	0.1
Mix 2: SP 19 6% RAS	30	7.9	8.7	8.9	8.5	0.6
Mix 3: SP 19 3% RAS and 25%RAP	35	8.7	7.9	8.6	8.4	0.4
Mix 4: SP12.5 FC1 3% RAS and 12%RAP	8	6.9	6.4	5.8	6.4	0.5
Mix 5: SP12.5 FC2 6% RAS	7	5.6	7.4	7.8	6.9	1.2
Mix 6: SP12.5 FC2 3% RAS and 12% RAP	10	6.8	6.4	5.9	6.3	0.5

3.3.3 Initial Testing of the Slabs

The surface texture and skid resistance tests were carried out on the slabs before running any freeze thaw cycles to determine the initial conditions of the pavement (slab) surface. The sand patch test was used for determining the slab surface texture while the British Pendulum was used to test for the skid number. This is an important aspect as it determines the safety parameter of the pavement on construction as well as helps in decision making on what kind of material to use on the pavement. Table 3-3 shows the surface texture classification [Meegoda 2009].

Pavement surface texture is a significant road surface feature that ultimately determines the tire-road interactions including the wet friction, tire-pavement noise, splash/spray, rolling resistance, tire wear, smoothness and hydroplaning potential [Meegoda 2009]. This is comprised of microtexture, macrotexture, megatexture and roughness. Surface texture may result from several differing factors such as material and construction properties hence it is an important characteristic to check for quality control purposes.

Table 3-3: Surface texture Classification [Meegoda 2009]

Texture Classification	Relative Wavelengths
Micro-texture	$\lambda < 0.5$ mm
Macro-texture	0.5 mm $\lambda < 50$ mm
Mega-texture	50 mm $\lambda < 500$ mm
Roughness	0.5 m $\lambda < 50$ m

3.3.3.1 ***Sand Patch Test***

Sand path test is used to visually quantify observations of differences in macrotexture. For this research, a 3.0mm^3 volume of micro glass beads passing sieve number 52 and retained on sieve number 100 was used for the sand patch test. The glass beads were placed on dry pavement surface and the bottom of the cylinder gently tapped to release any remaining particles. A rubber disc was then used to spread the glass beads in circular motion to form a circular pattern as shown in Figure 3-6. A millimetre rule was then used to measure the diameter at four different points and an average was used to determine the Mean Texture Depth (MTD) in mm [ASTM 1996]. Equation 3-3 was used for the evaluation of MTD;

$$MTD = \frac{4V}{\pi D^2}$$

Equation 3-3

Where; D – Average diameter of the area covered by the material (mm) and

V – Sample Volume (mm^3)

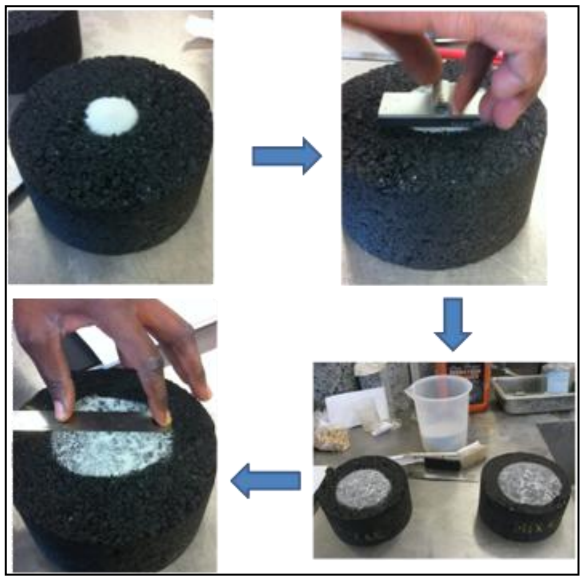


Figure 3-6: Sand Patch Method to Determine Surface Texture

3.3.3.2 ***Skid Resistance using the British Pendulum***

The Skid Resistance Value (SRV) or British Pendulum Number (BPN) was determined using the British Pendulum tester (Figure 3-7), which is a dynamic pendulum impact-type tester used to measure the energy loss when a rubber slider edge is propelled over a test surface [ASTM 1993]. The BPN attained represents the friction properties, which are then correlated with the skid number that indicates the safety quality of a pavement. The higher the skid resistance value, the higher the microtexture and the better the skid resistance.

It should be noted that the British Pendulum Test (BPT) is greatly depends on the workmanship of the operator

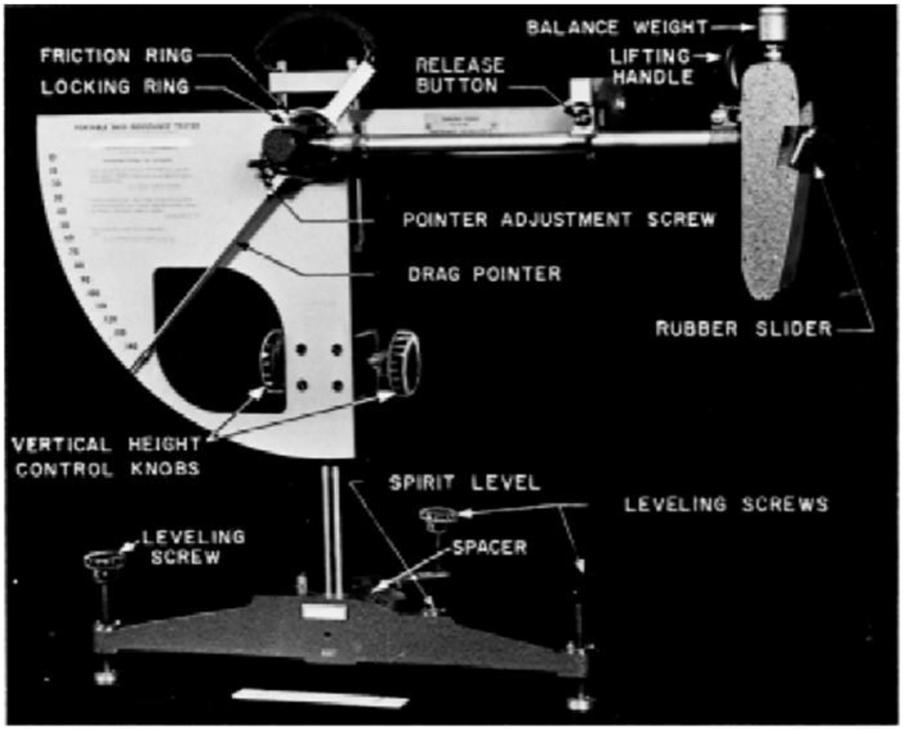


Figure 3-7: British Pendulum Tester [ASTM 1993]

The slab was placed in a static mould to reduce lateral movement during testing. The surface was cleaned to remove any loose particles, and leveled using a spirit level and three leveling screws on the pendulum base frame as demonstrated in Figure 3-8. The zero setting was then checked by raising the swinging arm to the horizontal position. The pointer was then brought round to its stop in line with the pendulum arm. The pavement surface was then sprayed with cold water using a spray bottle. The temperature of the water on the pavement surface was taken using a gun thermometer and recorded. The pointer was carried with the pendulum arm on the forward swung only to give the British Pendulum Number reading. The pendulum arm was then caught on its return swing and the pointer reading noted.

Prior to the swing, the sliding length of the rubber slider over the pavement surface under testing was checked by gently lowering the pendulum arm until the slider just touched the surface first on one side and then on the other side of the vertical. The sliding length (distance between the two points where the rubber touches the test surface) should be between 76mm for laboratory purposes while for field purposes, it is between 125mm to 127mm [RRL 1996]. Five successive reading were taken and an average used to calculate the BPN using Equation 3-4 [Oliver 1989].

$$Corrected\ BPN = \left({Mean\ BPN}\Big/{\left((1 - 0.00525) * (Temperature - 20)\right)} \right)$$

Equation 3-4

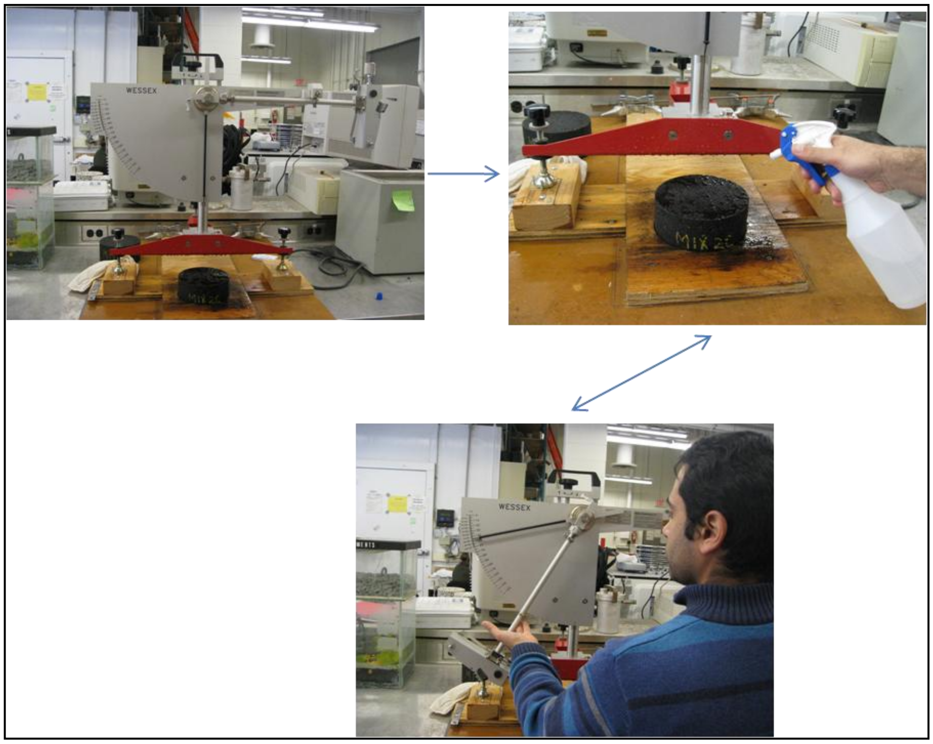


Figure 3-8: Skid Resistance Value Determination using the CPATT British Pendulum

3.3.4 Laboratory Freeze-Thaw Cycles

The weather data was attained from Environment Canada website. The Toronto Lester B. Pearson International Airport weather condition data was used to obtain the number of freeze-thaw cycles at temperatures, which could cause a damage effect to the pavement and were found to be approximately 76 cycles per year. This is a conservative value for a typical variation for a ten-year cycle but was deemed to be appropriate. The slabs was frozen for a maximum of 16 hours at 12.5^0C ± 2.5 and thawed for 8 hours at room temperature 21^0C.

3.4 Assessment of Field Test Sites

3.4.1 Pavement Distress Survey

The distress survey was carried out following the Ministry of Transportation Ontario (MTO) flexible pavement condition evaluation form to assess the condition of the pavement for both the RAS section at CPATT test track and the three residential streets in the Town of Markham [MTO 1989]. The attained assessment results were compared with the previous results carried out in Phase two in 2009 and 2010 for all test sections.

3.4.2 Pavement Deflection Measurement

Deflection measurements were performed on the CPATT Test Track using the CPATT lightweight deflectometer (LWD) Dynatest 3031 shown in Figure 3-9. Measurements were taken in the wheel path of both sides of the pavement and centreline every 50m along the pavement. The data was then downloaded from the Personal Digital Assistant (PDA) device also known as a palmtop computer (shown in Figure 3-10). The deflection measurements were then sorted out using Microsoft Excel program and an average of six readings was used. The deflection was then normalized to a standard stress distribution of 150kPa. A factor of two was used for the stress distribution and Poisson's ratio of 0.5 in accordance to standards [Dynatest 3031].

Any outliers in the stress distribution data were discarded because of lack of fit in the general trend observed. For comparison purposes, the deflection measurements were analyzed with the measurements attained in the preceding study in order to ascertain the rate of deterioration in the pavement condition under the expected traffic loading.

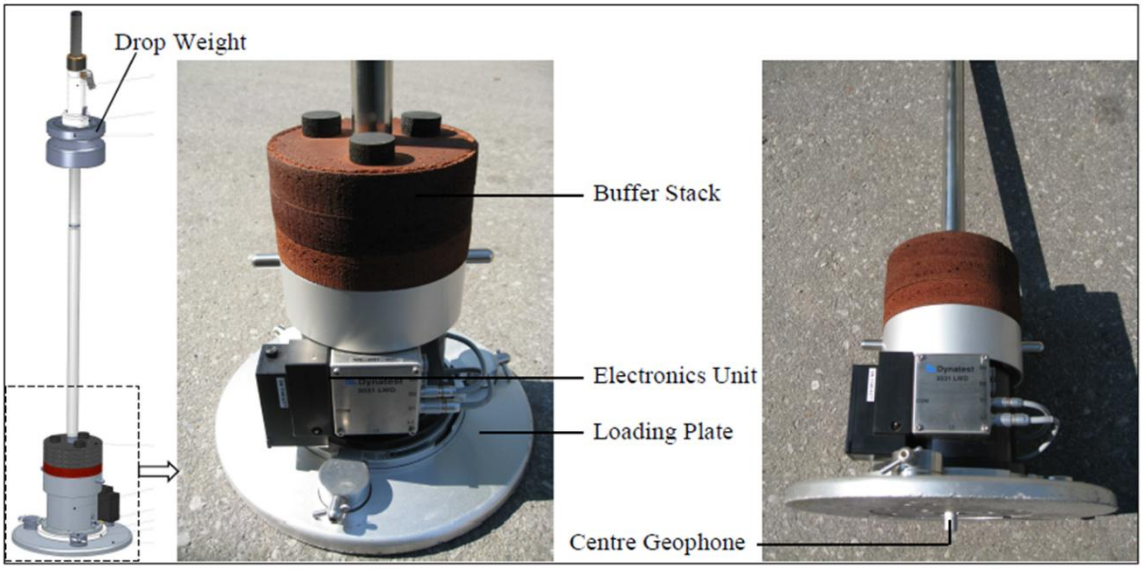


Figure 3-9: Light Weight Deflectometer (LWD), Dynatest 3031 [Du Tertre 2010]

The Dynatest 3031 instrument was used to measure deflection in the pavement. Calculations performed by the program of the equipment follows the elastic theory. The surface modulus for a homogenous, isotropic, linear-elastic half space, and static loading condition is defined as the weighted mean modulus of the pavement structure, given by Ullidtz 1987 work using Equation 3-5:

$$E_0 = \frac{f \cdot (1-\upsilon^2) \cdot \sigma_0 \cdot a}{d_0}$$

Equation 3-5

Where; σ_0 is the mean value of the stress on the surface, a is the radius of the loaded area, d_0 is the deflection measured with the centre geophone, υ is the Poisson's ratio and f is a factor that depends on the stress distribution:

- Uniform: $f = 2$
- Rigid plate: $f = \pi/2$

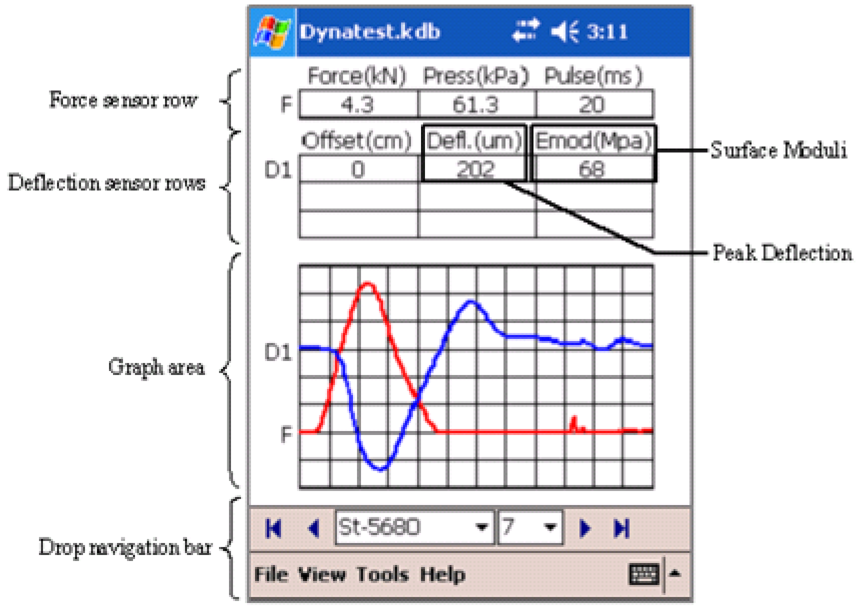


Figure 3-10: Dynatest 3031 LWD – PDA Display [Dynatest 3031]

3.5 Pavement Computer-Based Performance Analysis

3.5.1 Mechanistic-Empirical Pavement Design Guide (MEPDG)

The MEPDG, a pavement design guide developed by AASHTO under the U.S. National Cooperative Highway Research Program (NCHRP) project 1-37A, utilizes the mechanistic-empirical principles to predict pavement deteriorations and their expected service lives. This protocol is also being calibrated for use in Canada [Mills 2007]. The guide is very comprehensive and includes procedures for the analysis and design of new and rehabilitated rigid and flexible pavements, evaluating existing pavements, sub-drainage design procedures, rehabilitation treatments recommendations, foundation improvements as well as life cycle cost analysis [ARA 2011].

The MEPDG guide depends heavily on the characterization of the fundamental material engineering properties, which requires four sets of input data namely; traffic, environmental/climatic influences, material, and pavement response and distress models. The design criteria also account for the various climatic influences that may cause pavement response. Different design scenarios and analyses using combinations of RAS and/or RAP were carried out to determine expected performance. If a criterion did not meet the necessary performance level, it was modified and reanalysed until satisfactory results were achieved.

Level 1 is the most accurate level usually used on major highways, interstate, and strategic roads. Sometimes it is used where specific material characterization is required. Level 2 is the medium accuracy input level used when there is limited testing of the material used in the pavement layers while level 3 represents the least accuracy level used on low volume roads. Figure 3-11 shows the schematic flow of pavement design and performance analysis processes.

All analyses in this study were carried out using Level 1 which is the most data intensive and provides the highest level of accuracy with less error.

Level 1 Design:

The design required project specific inputs such as material properties measured using laboratory testing and binder properties for HMA. Dynamic modulus test was carried out on four samples per HMA mix and all four dynamic modulus results analysed for any abnormalities, and then an average value was used for the MEPDG run.

The base/subbase and subgrade were designed according to the Ontario Provincial Standard Specification (OPSS) 1010 [MTO 2004] for Eastern and Southern Ontario. Table 3-4, Table 3-5, and Table 3-6 present the design inputs used in the MEPDG for performance analysis. Poison’s ratio of 0.35 and Coefficient of Lateral Pressure (K_0) of 0.5 were used at all pavement layers.

HMA E* Predictive Model; NCHRP 1-37A Viscosity based Model, HMA Rutting Model Coefficient; NCHRP 1-37A, Coefficient and Endurance Limit (Microstrain); 100 were used in the design for performance prediction.

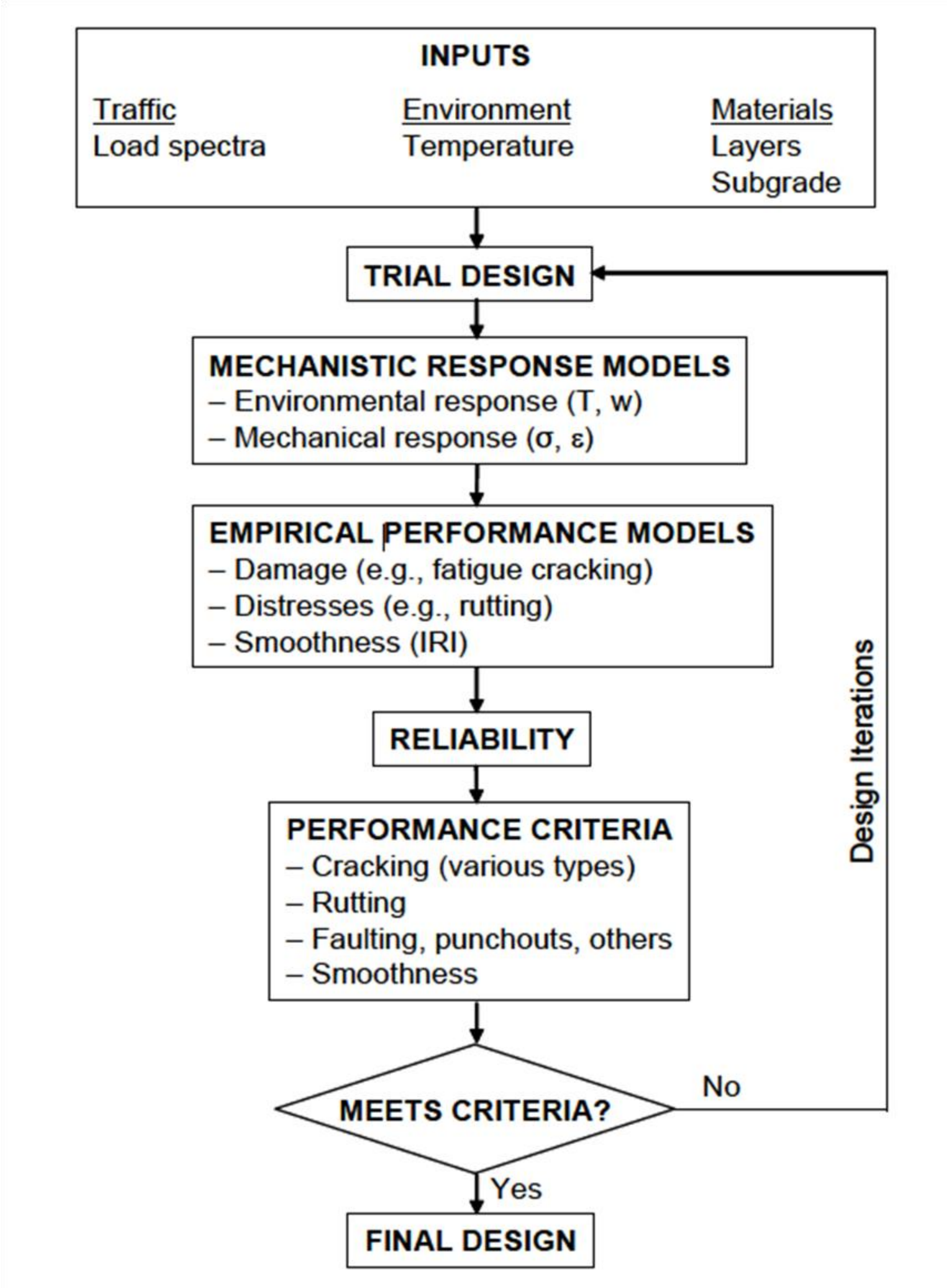


Figure 3-11: Schematic Diagram for Mechanistic-Empirical Pavement Design Methodology [Schwartz, 2007]

Table 3-4: MEPDG Performance Criteria and Design Inputs

Pavement Layer	Pavement Design Description and Requirements
General Inputs	• One Kilometre Lane (0.62 miles) • Design Life – 20 years • Reliability Level – 70% • Operational Speed – 60Km/hr (37.28Mile/hr) • Air Voids – 7% • Climate Data – Toronto Lester B. Pearson International Airport **Terminal Serviceability Limits** • Fatigue (Alligator) Cracking – 25% • Longitudinal Cracking – 10560m/km (2000ft/mile) • Thermal (Transverse) Cracking – 5575.68m/Km (1056 ft/mile)[1] • Rutting – 19.05mm (0.75 inches) • International Roughness Index (IRI) – 3.0mm/m (190 in/mile)
Traffic	• 2500 AADTT, Growth Rate of 2% • Number of lanes – 1 in each direction • Percent trucks in design lane – 50% • Percent trucks in design lane – 100%
Hot Mix Asphalt Layer	• Surface layer thickness – 90mm (3.5 inches) • Dynamic Modulus, E* (Laboratory values) • Binder Properties (Asphalt Binder Complex Shear Modulus, G* (Pa) and Phase angle, Ø (degree))
Base	• Granular Base A – 150mm (6 inches) • Crushed Stone, less sand
Subbase	• Granular Base B – 450mm (18 inches) • Crushed Stone with sand
Subgrade	• GW – Well graded gravel

[1] Thermal Cracking for all the mixes produced abnormal results hence it was neglected in the analysis.

With the MEPDG, a state-of-the-practice mechanistic model was created to predict the accumulated pavement distresses over time due to the traffic loading and the material properties. A summary of MEPDG inputs per mix is given in Table 3-5 as derived by the program in respect of the entered mix characteristic details. Due to addition of more than 20% Recycled Asphalt Pavement (RAP) by mass to the mixes, the final result showed an increase in PG grade by one grade. The MEPDG runs were carried out at a grade higher for each mix as encouraged by MTO OPSS 1150 [MTO 2007].

Table 3-5: MEPDG Inputs for Asphalt Concrete (AC) Layer

Mixture Characteristics	**Mix 1**	**Mix 2**[1]	**Mix 3**	**Mix 4**	**Mix 5**[1]	**Mix 6**
	HL 3: 1.5% RAS, 13.5% RAP	**SP 19: 6% RAS**	**SP 19: 3% RAS, 25% RAP**	**SP 12.5 FC1: 3% RAS, 17% RAP**	**SP 12.5 FC2: 6% RAS**	**SP 12.5 FC2: 3% RAS, 12% RAP**
PG Grade – Mix Design	PG 58-28	PG 52-40	PG 52-34	PG 52-34	PG 52-40	PG 52-34
PG Grade – MEPDG Analysis	PG 64-22	PG 58-40	PG 58-28	PG 58-28	PG 58-40	PG 58-28
Volumetric Properties as Built (MEPDG Default)						
Effective Binder Content (%)	11.6	11.6	11.6	11.6	11.6	11.6
Air Voids (%)	7	7	7	7	7	7
Total Unit Weight (pcf)	150	150	150	150	150	150
Thermal Properties (MEPDG Default)						
Thermal Conductivity Asphalt (BTU/hr-ft-F^0)	0.67	0.67	0.67	0.67	0.67	0.67
Heat Capacity Asphalt (BTU/lb-F^0)	0.23	0.23	0.23	0.23	0.23	0.23
Thermal Cracking Properties						
Average Tensile Strength at 14^0F	415.56	583.69	388.87	388.87	583.69	388.87
Mixture VMA (%)	18.6	18.6	18.6	18.6	18.6	18.6
Aggregate Coefficient Thermal Contraction (in/in)	0.000005	0.000005	0.000005	0.000005	0.000005	0.000005
Aggregate Coefficient Thermal Contraction (in/in/^{0}F)	0.000013	0.000013	0.000013	0.000013	0.000013	0.000013

[1] PG 58-34 could not run with the achieved laboratory dynamic modulus of Mix 2 and Mix 5; therefore PG 58-40 was used.

Aggregate and Soil Gradation

In Ontario, Granular A/base and Granular B/subbase are the most commonly used aggregates in pavement construction beneath the HMA pavements, and these are well described in OPSS 1010 [MTO 2004]. The subgrade is an important component of pavement design and therefore the selection of appropriate properties should be thoroughly explored before actual construction. For the study, a well graded gravel soil (GW) was chosen for the analysis of pavement performance adopted for the province of Ontario, Canada. The gradations and material properties used in the analysis are given in Table 3-6 and Table 3-7.

Table 3-6: MEPDG Inputs for Granular layers and Subgrade Layer

Material Property	Granular A	Granular B	Subgrade
Maximum Dry Unit Weight (pcf)	127.7	127.6	128.4
Specific Gravity of Solids, Gs	2.7	2.7	2.7
Saturated Hydraulic Conductivity (ft/hr)	18.34	0.00803	0.08468
Optimum Gravimetric Water Content (%)	7.4	7.4	7.0
Calculated Degree of Saturated (%)	62.3	62.5	60.6
Plasticity Index[2]	1	1	2
Liquid Limit[2]	6	6	8
Modulus	[1]250 MPa	[1]200 MPa	[2]275.79MPa
Poisson's Ratio	0.35	0.35	0.35
Coefficient of Lateral Pressure (K0)	0.5	0.5	0.5

Note: ***[1]*** Ontario Provincial Standard Specification, MTO 2004, ***[2]*** MEPDG Default modulus values for Granular A, Granular B, and GW) as well as plasticity and liquid limit indices. N/A – Not Applicable.

Table 3-7: Material Gradation (Granular and Subgrade Layers)

Material Property	Aggregate Gradation (Percent Passing)		Soil Gradation (GW)
	Granular A	Granular B	
Gradation 150 mm	100	100	100
Gradation 106 mm	100	100	100
Gradation 26.5 mm	100	100	100
Gradation 19.0 mm	85	N/A	94
Gradation 13.2 mm	65	N/A	80.3
Gradation 9.5 mm	50	N/A	64.4
Gradation 4.75 mm	35	55	43.3
Gradation 1.18 mm	15	40	29.7
Gradation 300 µm	5	22	14.1
Gradation 150 µm	N/A	N/A	7.8
Gradation 75 µm	2	10	4.18

3.5.2 Life Cycle Cost Assessment (LCCA)

The primary purpose of LCCA is to evaluate long-term economic implications of initial pavement decisions such as the use of recycled material such as RAS and/or RAP in HMA pavement as compared to conventional pavements. Through the LCCA technique, decision makers/designers are able to identify the lowest cost alternative that accomplishes the project objectives without compromising the pavement performance or posing any constructability and maintainability issues as well as environmental effects by providing all the critical information. The LCCA procedure performed in the study is given below:

1. A 20-year analysis period was assumed.
2. Alternative strategies over the analysis period of 20 years were identified. This was based on the previous studies carried out for the Ministry of Transportation Ontario in 1998, 2006 and 2011. The rehabilitation and maintenance strategies developed were also established with their expected life span.
3. Agency unit cost estimates for pavement material, initial construction, expected maintenance and rehabilitation plans, labour, user and non-user costs were calculated from MTO LCCA reports. The Present Value (PV) calculation puts into consideration the discount rate and the time a cost was/will be incurred in order to establish the present value cost of the base year of the analysis period. The PV acts as an equalizer summing up the initial and future costs. The unit costs used in the study are as given in Table 3-8 and Table 3-9 as obtained from a 2011 Ontario report [ARA 2011].

Table 3-8: Initial Flexible Pavement Construction for Average Annual Daily Truck Traffic (2500 AADTT)

Pavement Layer	Description of Pavement Layer, Amount (Quantity)	Unit Cost
Hot Mix Asphalt (HMA) (tons)	Superpave 12.5 FC2	$120
	Superpave 12.5 FC1	$115
	Superpave 19	$96
Base	Granular A (t)	$18
Sub-base	Granular B (t)	$15
Subgrade	Earth Excavation (m^3)	$18

** 1yd^3 is equivalent to 0.766 m^3 while 1yd^3 is 0.27 tons

Table 3-9: Pavement Maintenance and Rehabilitation Treatment Plans for Average Annual Daily Truck Traffic (2500 AADTT)

Years after Initial Construction	Service Life	Description of Pavement Layer, Amount (Quantity)	Quantity (per 1km of road)	Unit Cost
3	5-7	Rout and Seal (m)	200	$5
5	8-10	Spot Repairs, mill 40mm/patch 40mm, 5% area (m^2)	750	$35
9	8-10	Spot Repairs, mill 40mm/patch 40mm, 5% area (m^2)	750	$35
15	5-7	Rout and Seal Crack (m)	200	$5
15	8-10	Spot Repairs, mill 40mm/patch 40mm, 20% area (m^2)	3,000	$35
20	10-12	Mill 90mm/Place 90mm Asphalt Pavement (Overlay)	3,375	$15
20	10-12	Resurface with Superpave 19, 50mm (t)	1845	$96
20	10-12	Resurface with Superpave 12.5 FC1, 40mm (t)	1512	$115
20	10-12	Resurface with Superpave 12.5 FC2, 40mm (t)	2288	$120

The salvage value (representing the value of the investment alternative at the end of the analysis period) was also calculated based on the cost of the final rehabilitation activity, expected life of rehabilitation, and time since last rehabilitation activity with Equation 3-6. Salvage value can either be negative (indicating a value associated with the pavement at the end of the analysis period) or positive (indicating that there are disposal costs with the pavement at the end of the analysis period) [Demos 2006].

$$SV = 1 - \left({}^{L_A}/_{L_E} \right) * C$$

Equation 3-6

Where; C – cost of rehabilitation strategy

L_A – portion of expected life consumed

L_E – expected life of the rehabilitation strategy

4. An expenditure stream was developed in the form of graphical/tabular representation of expenditure over the analysis period for each pavement design strategy.

5. The net present value (NPV) was computed; the LCCA evaluates the cost efficiency of the investment strategies by discounting the future cost to the base year that is then added to the initial cost. Both the user and agency costs were incorporated into the analysis. The user costs are the delay, vehicle operating and crush costs incurred by the user of the facility. The NPV as computed in Equation 3-7 is the economic efficiency indictor of choice.

$$NPV = Initial\ Cost + F\left(\frac{1}{(1+i)^n}\right)$$

Equation 3-7

Where; F – Future cost at the end of the n^{th} year

i – Discount rate

n – Number of years

6. The results were then analyzed through sensitivity analysis to determine the influence of the major input variables such as the design life. This analysis also known as the risk analysis is a probabilistic approach incorporated in the LCCA to address the variability within the major analysis input assumptions and estimates. This is done by evaluating the different discount rates or assigned time values hence estimating the best and worst case scenarios.

7. Once the NPV had been calculated for each of the design strategies, they were reevaluated for best possible options that did not compromise pavement performance, workability, and the environment at the appropriate economical cost. An Equivalent Uniform Annual Cost (EUAC) was used for the assessment using Equation 3-8. EUAC represents the NPV of all the discounted costs and benefits of an alternative as if they were to occur uniformly throughout the analysis period.

$$EUAC = NPV\left(\frac{1(1+i)^n}{(1+i)^n - 1}\right)$$

Equation 3-8

3.5.3 Environmental Analysis using PaLATE

Pavement Life Cycle Assessment Tool for Environment and Economic Effect (PaLATE) tool is a computer-based life-cycle spreadsheet developed by Horvath of the University of California at

Berkeley, assists in decision making by evaluating the use of pavement materials (both virgin and recycled construction materials) in highway constructions including both life cycle cost and environmental impacts parameters. The PaLATE tool was downloaded Horvath's website [Horvath 2003].

According to PaLATE, the tool assesses the environmental and economic feasibility of pavement recycling compared to the virgin material usage by analyzing the user inputs for the design, initial construction, maintenance, equipment use, and pavement costs to provide outputs for environmental effects and life-cycle costs [Horvath 2003]. In order to estimate the environmental effects of the material used by the pavement, the tool requires the pavement thickness design (this illustrates the length, width, and depth), material breakdown, and material haul distance.

For this study; all PaLATE workbooks assumed a one-kilometre (0.621miles) pavement length. The pavement width and thickness were based on the CPATT test track design. A one-lane pavement with lane width of 4m and depth of 90mm (3.5 inches) as shown in Figure 3-12 with the granular base and granular subbase as used for the test track cross section.

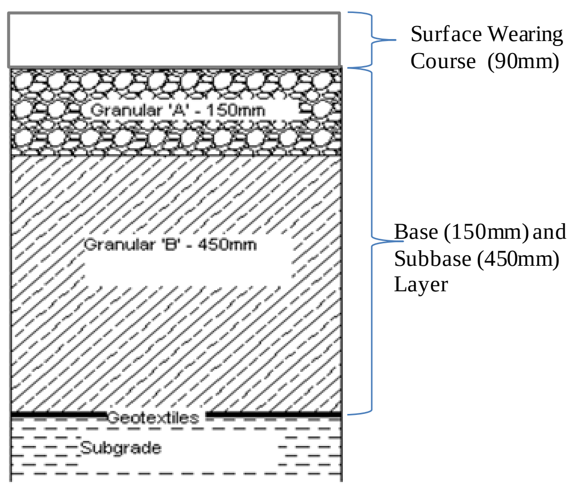


Figure 3-12: Cross Section of the CPATT Test Track RAS Section

The PaLATE tool requires the use of volumetric proportions of the materials per pavement layer and the assumptions used in the analysis and proportions are in volume. According to guidelines developed by Asphalt Pavement Association of Indiana, for equivalency RAP calculation, 1% roofing shingles can be counted as equivalent to 5% RAP hence the total quantity of virgin aggregates used in the mix can be calculated according to how much recycled material was present in the mix design [APAI 2009].

The PaLATE tool requires the haul distances from the contractor's site to the construction site. The distances used in this study were articulated according to practical distances from where the materials in the mixes were obtained. The materials were acquired from two supplier companies and the distance is calculated based on University of Waterloo as a construction site, namely:

- Asphalt Binder, McAsphalt Industries Limited (115km ≈ 72 miles)
- Virgin Aggregates, Recycled Asphalt Pavement (RAP), Sand, and Shingles
- Miller Paving Limited (121km ≈ 75 miles)

Quantities of virgin aggregate, asphalt binder, and RAP for the mixes were calculated following the design mix provided by Miller Paving Inc using the densities given in Table 3-10. All the densities used in the research were as reported in PaLATE except for RAS which was calculated from Equation 3-9. The equation puts into consideration the asphalt binder and aggregate which make up over 95% shingles [Austin 2011].

$$\rho_{RAS} = \rho_{Binder} V_{Binder} + \rho_{Aggregate} V_{Aggregate}$$

Equation 3-9

$$\boldsymbol{\rho}_{\mathrm{RAS}} = 0.84 \mathrm{x} 30\% + 1.85 \mathrm{x} 65\% = \mathbf{1.7\ tons/yd^3}$$

The RAS density lies within the densities reported by IOWA Department of Transportation, maximum specific gravity of 1.623 equivalent to 1.367 tons/yd^3 and effective specific gravity of 2.108 equivalent to 1.78tons/yd^3 [Seal 2010].

Table 3-10: Density Used in PaLATE Analysis [Horvath 2003]

Material	Suggested Density (tons/yd^3)	Density used (tons/yd^3)
Asphalt Mixture	1.23	2.16*
Asphalt Binder	0.84	0.84
RAP	1.62 - 1.89	1.85
RAS	1.37 - 178	1.7
Virgin Aggregate	1.25	2.23*

[*] Adopted from Austin 2011

3.6 Statistical Analysis of Experimental Results

3.6.1 Introduction

A statistical approach was employed in the research to investigate the differences and determine if results were statistically significant. The analysis investigated the effect of the dependent variables (HMA Mixtures) on the independent variables (structural loading, environment/climate, and cost). An experimental study was used to evaluate the HMA mix performance while an observational study was used to investigate the sustainability of the HMA mixes. The F-test, T-test and Analysis of Variance (ANOVA) were employed in the research to investigate the variation of the six alternative mixes from the control, conventional HL 3 to examine the statistical differences between the mixes.

The null hypothesis (H_o) used the control HL 3 variable to test if there was any relationship between the measured variables and the mixes that contain RAS and/or RAP. It is paired with the second hypothesis; the alternative hypothesis (H_1) which examines variability of the alternative mixes [Montgomery 2001].

H_o: $\mu_1 = \mu_2$ – Fail-to-reject the null hypothesis (both mixes are consistent in performance)

H_o: $\mu_1 \neq \mu_2$ – Reject the null hypothesis

However, two types of errors can occur when testing the hypotheses, Type I error (if null hypothesis is rejected when it is true) and Type II error (if null hypothesis is not rejected when it is false). The general procedure in hypothesis testing lies in specifying a value probability of Type I error α often referred to as significance level of the test [Montgomery 2001].

The significance level (α) or confidence level (%) determines the degree of evidence at which the difference/variability in the variables is unlikely to have arisen by chance. A 95% confidence level ($\alpha = 0.05$) was used in the study. The level of significance and degree of freedom were used to read the critical values from both the F-test and T-test tables.

Analysis of Variance (ANOVA), a statistical tool used for measuring the relative difference between means for different data sets, was used to analysis HMA mixture performance and sustainability characteristics.

The ANOVA table was used in testing of the laboratory results obtained during freeze-thaw cycling. This helped to assess if there was a significant change in the surface texture characteristics and skid resistance properties during and after the simulated two years in service.

3.6.2 F-Test Analysis

An F-test, a statistical test comprising of F-distribution under the null hypothesis test statistics, was used to compare the statistical models with fitting data sets. The test is designed to determine if two population variances are equal by comparing the two variances and identify the best model which best fits the population (in this case designed mixes with better performance, sustainability, and cost) using least squares method. The F-distribution is non-negative and non-symmetrical distribution. The hypothesis testing was performed on assumption that the null hypothesis was true meaning the control mixes (mixes in service) performed better than the five alternative mixes in all aspects. The F-test considers the variability in terms of sum of squares reflecting the different source of variation. The sum of squares (SS) tends to be greater when the null hypothesis is not true hence SS have to be statistically independent for the F-distribution under null hypothesis to follow. F-value was calculated as shown in Equation 3-10.

$$F_{Calculated} = \frac{S^2{}_{Control\ Mix}}{S^2{}_{Alternative\ Mix}}$$

Equation 3-10

Where S^2 – Variance of either control mixes or alternative mixes

If the $F_{Calculated} > F_{Critical}$, the H_o is rejected concluding that there were differences in the HMA mixes and it is in favor of the alternative mix whereas if $F_{Calculated} < F_{Critical}$, a weak conclusion could be drawn or indicates lack of statistical significant evidence of variation. In this case the control and alternative mixes are statistically observed to be consistent with each other.

The F-Test was used in validating the performance prediction models to determine if there was any significant change in performance if recycled material is added to HMA mixes. It also helped to assess consistencies in data from the field.

3.6.3 T-Test Analysis

This is a statistical hypothesis test following the T-distribution and/or normal distribution (when value of the scaling term in the test statistics is known). The null hypothesis test was used to study the difference between the responses (by the design mixes) on the same statistical unit assuming the mean value was zero. An independent one-sample T-test was employed for the study whereby the null hypothesis was tested to examine whether the alternative mix mean (μ_1) was equal to the control mix mean (μ_o) and t-value was calculated using Equation 3-11.

$$t_{Calculated} = \frac{(\mu_1 - \mu_o)}{S/\sqrt{n}}$$

Equation 3-11

Where; μ_o – Control Mix Mean, μ_1 – Alternative Mix Mean

S – Standard Deviation, n – Sample Size

If the $t_{Calcualted} > t_{Critical}$, the null hypothesis was rejected concluding that there was difference in the mixes and that the alternative mix was better suited for use in hot-mixed asphalt pavement type whereas if $t_{Calcualted} < t_{Critical}$, then we fail to reject the null hypothesis. This indicates a weak statistical evidence of difference or lack of significant statistical difference in the mixes hence strong evidence of consistence of the alternative mix with the control mix.

The T-Test was used in testing the significance of increasing the percentages of RAS and/or RAP in HMA mixes.

3.7 Summary

The methodology used in this research was presented in this chapter. This included the approach taken to evaluate the performance of HMA pavements with certain amounts of RAS, life cycle assessment of the material used as well as laboratory and field condition assessment of the pavements. Life cycle assessment included both the economic and environmental impact assessments. A statistical analysis for the validation of the performance and sustainability models is also discussed in the chapter.

Chapter 4: Laboratory and Field Pavement Evaluation

4.1 Introduction

This chapter discusses both the field and laboratory pavement condition assessment carried out on asphalt pavements incorporated with RAP and/or RAS. The laboratory asphalt slab evaluations simulate the effect of the environmental natural conditions by running complete year freeze-thaw cycles. The field evaluations were carried on pavements that have been subjected to natural conditions such as traffic loading, pavement aging, and environmental effects due to climatic changes, this simulates the rate of deterioration of asphalt pavement with RAS.

Pavement performance is represented by the pavement deterioration process, which is a complex process involving not only structural fatigue but also many functional pavement distresses as a result of interaction with traffic, climate, material, and age [Mohd Isa 2005]. The pavement condition evaluation consists of visual rating for transverse cracking, longitudinal cracking, as well as smoothness, rutting performance, and surface characteristics. The results were reported as specified by the Ministry of Transportation Ontario (MTO) pavement condition evaluation guidelines.

4.2 Laboratory Testing at CPATT

Initial surface texture and friction testing as well as visual surface assessment were carried out on the slabs prior to freeze-thaw cycling to ascertain the initial conditions of the pavement surface and/or structure. The asphalt slabs were observed to be in excellent condition and did not exhibit any cracks as shown in Figure 4-1.

The asphalt slabs then underwent 152 cycles of freeze-thaw to simulate two full years of an asphalt pavement in service. Surface texture, friction test, and visual slab distress assessments were carried out at the end of each full cycle to deduce the rate of deterioration. The first and second full cycles were tested on May 9th, 2011 and August 2nd, 2011 respectively. The slabs were weighed immediately after the cycles to determine their mass and measured in height and diameter to determine if there were any changes due to freeze-thaw cycling.

Figure 4-1: Initial Asphalt Slab Properties

4.2.1 Physical Property Evaluation

4.2.1.1 *Initial Surface Texture and Surface Distresses*

Although visually, the pavement slab surface texture was characterized by their rough texture, and unevenness nature except for Mix 4 (SP 12.5 FC1 3% RAS and 17% RAP), the test indicated that the texture was composed of fine aggregates which were smooth. This was crosschecked with the texture classification given in Table 3-3. Table 3-3 The initial surface texture depth is given in Table 4-1. All slabs were less than 5mm in texture depth indicating that the surface was smooth and dominated by microtexture properties.

Table 4-1: Initial Surface texture of the Asphalt Slabs

Sample	Number of Diameter Measurements (mm)				Average Diameter (mm)	Volume (mm^3)	Mean Texture Depth (mm)
	1	2	3	4			
Mix 1: HL 3A	9.8	8.5	9.0	9.5	9.2	3.00	0.05
Mix 1: HL 3B	9.0	8.5	9.2	8.2	8.7	3.00	0.05
Mix 1: HL 3C	10.4	8.3	9.2	9.2	9.3	3.00	0.04
Mix 2A	10.1	9.1	8.2	8.9	9.1	3.00	0.05
Mix 2B	9.9	9.1	10.2	10.2	9.9	3.00	0.04
Mix 2C	10.5	9.3	9.9	9.5	9.8	3.00	0.04
Mix 3A	10.6	9.3	10.3	10.3	10.1	3.00	0.04
Mix 3B	10.2	10.0	10.0	9.7	10.0	3.00	0.04
Mix 3C	9.6	9.2	9.4	9.3	9.4	3.00	0.04
Mix 4A	10.3	9.8	10.5	9.9	10.1	3.00	0.04
Mix 4B	10.9	9.3	10.5	10.2	10.2	3.00	0.04
Mix 4C	9.9	10.6	10.2	10.4	10.3	3.00	0.04
Mix 5A	8.9	6.7	7.7	8.3	7.9	3.00	0.06
Mix 5B	8.8	8.4	8.0	8.2	8.4	3.00	0.05
Mix 5C	8.0	7.3	7.4	7.3	7.5	3.00	0.07
Mix 6A	8.0	7.9	8.5	7.0	7.9	3.00	0.06
Mix 6B	7.5	6.7	7.0	7.0	7.1	3.00	0.08
Mix 6C	9.6	8.7	8.7	9.0	9.0	3.00	0.05

4.2.1.2 *After One Year (First Set of Freeze-Thaw Cycles): Physical Properties*

The slabs exhibited a slight decrease in diameter for all the mixes except Mix 2 from the original construction diameter of 150mm. However, from the statistical analysis, it is observed that there was no significant statistical change in diameter of the slabs. Hence, it was consistent with the original diameter. Mixes 1, 2, 3, 4, and 5, demonstrated a decrease in height except for Mix 6, which was observed to have increased in height. The statistical analysis indicates that there was a significant difference in height of the slabs after the first set of freeze-thaw cycles for all the mixes. Generally, all the mixes increased in mass, this is supported by the statistical analysis that indicated significant change in mass of the slabs. Overall, there was a slight change in the physical properties of the slabs as given in Table 4-2 and Table 4-3.

Table 4-2: Physical Properties of the Samples (After One Year)

					First Freeze-thaw Cycle			Difference		
Mix Description	Slab Number	Initial Diameter (mm)	Initial Height (mm)	Initial Mass (g)	1st Cycle Diameter (mm)	1st Cycle Height (mm)	1st Cycle Mass (g)	Diameter (mm)	Height (mm)	Mass (g)
Mix 1: HL 3 1.5% RAS and 13.5% RAP	A	150.0	76.8	2983.5	149.9	72.9	2985.1	0.1	4.0	-1.6
	B	150.0	73.0	2988.6	150.1	73.5	2990.4	-0.1	-0.5	-1.8
	C	150.0	73.6	3001.2	150.0	73.1	3003.0	0.0	0.5	-1.8
Mix 2: SP 19 6% RAS	2A	150.0	74.0	2995.1	149.9	73.9	2994.5	0.1	0.1	0.6
	2B	150.0	74.2	3003.6	149.9	73.9	3006.0	0.1	0.3	-2.4
	2C	150.0	73.8	2983.0	150.0	73.5	2985.3	0.0	0.3	-2.3
Mix 3: SP 19 3% RAS, 25% RAP	3A	150.0	73.8	2996.6	150.0	73.5	2999.0	0.0	0.3	-2.4
	3B	150.0	73.5	2995.5	150.0	73.4	2997.7	0.0	0.2	-2.2
	3C	150.0	73.6	2994.0	150.1	73.6	2996.0	-0.1	0.1	-2.0
Mix 4: SP12.5 FC1 3% RAS, 12% RAP	4A	150.0	72.5	2994.5	150.1	72.4	2996.6	-0.1	0.1	-2.1
	4B	150.0	71.9	2991.2	150.1	71.8	2993.3	-0.1	0.2	-2.1
	4C	150.0	71.6	2995.8	150.1	71.6	2997.7	-0.1	0.0	-1.9
Mix 5: SP12.5 FC2 6% RAS	5A	150.0	72.3	2993.6	149.9	71.8	2997.4	0.1	0.5	-3.8
	5B	150.0	74.2	2995.2	150.0	73.4	2997.5	0.0	0.7	-2.3
	5C	150.0	74.5	3002.5	150.2	74.1	3004.2	-0.2	0.4	-1.7
Mix 6: SP12.5 FC2 3% RAS 12% RAP	6A	150.0	72.0	3045.3	150.7	73.3	3048.9	-0.7	-1.3	-3.6
	6B	150.0	72.0	3003.4	150.4	72.3	3006.8	-0.4	-0.3	-3.4
	6C	150.0	72.2	2995.9	150.2	72.0	2999.8	-0.2	0.2	-3.9

The $F_{Calculated} < F_{Critical}$ for diameter and the $F_{Calculated} > F_{Critical}$ for height and mass, which demonstrated that the diameter was statistically consistent throughout the year cycle while the height and mass seemed to generally decrease and increase, respectively as given in Table 4-3. The physical properties of the asphalt pavements were observed to be slightly affected by the climatic changes. However, the slabs were observed to hold on well to the freeze-thaw cycles simulating the climatic changes in Canada.

Table 4-3: After One Year Physical Properties Statistical Analysis

Description	***Initial Diameter (mm)***	***1st Cycle Diameter (mm)***	***Initial Height (mm)***	***1st Cycle Height (mm)***	***Initial Mass (g)***	***1st Cycle Mass (g)***
Mean	150	150.09	73.31	72.99	2997.69	2999.96
Variance	0	0.03	1.64	0.67	175.16	185.74
Observations	18	18	18	18	18	18
degree of Freedom (df)	17	17	17	17	17	17
F Calculated	**0**		**2.46**		**0.94**	
P(F<=f) one-tail	**0**		**0.04**		**0.45**	
F Critical one-tail	**0.44**		**2.27**		**0.44**	

The surface texture was consistent with the previously tested surface texture; hence, there was no statistical change in surface texture of the slabs illustrated by $F_{Calculated} < F_{Critical}$. This is given in Table 4-4 and Table 4-5. However, from Table 4-4, it was observed that there was a 0.01 decrease in mean texture depth of the pavement surface but this is very slight change. Therefore; it can be concluded that pavements constructed with the six mixes are expected to be in good condition at the end of the first year of construction when all climatic changes are taken into consideration.

Table 4-4: After One Year (First Set of Freeze-Thaw Cycles) Surface Texture

Mix Description	Slab Number	Initial MTD (mm)	1st Cycle MTD (mm)	Difference
Mix 1: HL 3 1.5% RAS and 13.5% RAP	A	0.05	0.04	0.01
	B	0.05	0.04	0.01
	C	0.04	0.04	0.01
Mix 2: SP 19 6% RAS	2A	0.05	0.05	0.00
	2B	0.04	0.04	0.00
	2C	0.04	0.04	0.00
Mix 3: SP 19 3% RAS and 25%RAP	3A	0.04	0.04	0.00
	3B	0.04	0.04	0.00
	3C	0.04	0.03	0.01
Mix 4: SP12.5 FC1 3% RAS and 12%RAP	4A	0.04	0.03	0.01
	4B	0.04	0.03	0.00
	4C	0.04	0.04	0.00
Mix 5: SP12.5 FC2 6% RAS	5A	0.06	0.07	-0.01
	5B	0.05	0.05	0.00
	5C	0.07	0.06	0.01
Mix 6: SP12.5 FC2 3% RAS and 12% RAP	6A	0.06	0.05	0.01
	6B	0.08	0.07	0.01
	6C	0.05	0.04	0.00

Table 4-5: After One Year Surface Texture Statistical Analysis

Description	***Initial MTD (mm)***	***1st Cycle MTD (mm)***
Mean	0.05	0.04
Variance	0.00	0.00
Observations	18	18
degree of Freedom (df)	17	17
F Calculated	**1.02**	
P(F<=f) one-tail	**0.49**	
F Critical one-tail	**2.27**	

4.2.1.3 ***After Second Year (Second Set of Freeze-Thaw Cycles): Physical Properties***

A visual survey carried on the asphalt slabs after the second set of freeze-thaw cycles indicated that they were still in good condition without any significant distresses. The slabs were observed to decrease in height and increase in mass with no significant change in diameter given in Table 4-6. The statistical analysis performed demonstrated the change in the physical characteristics of the slabs. There was no statistically significant change in diameter or change in height as demonstrated by F-test in Table 4-7. The $F_{calculated} < F_{Critical}$ and P-Value < 0.01 indicates strong evidence of consistence with the initial diameter and height.

However, there was a statistically significant change in the mass of the slabs after the second set of freeze-thaw cycle was completed as given in Table 4-7. The $F_{calculated} > F_{Critical}$ and P-Value > 0.1 indicates weak statistical evidence. In general, the physical properties of the slabs were observed to be slightly effected by the second freeze-thaw cycling by the climatic changes.

Table 4-6: Physical Properties of the Samples (After Second Year)

Mix Description	Slab Number	Initial Diameter (mm)	Initial Height (mm)	Initial Mass (g)	Second Freeze-thaw Cycle: 2nd Cycle Diameter (mm)	Second Freeze-thaw Cycle: 2nd Cycle Height (mm)	Second Freeze-thaw Cycle: 2nd Cycle Mass (g)	Difference: Diameter (mm)	Difference: Height (mm)	Difference: Mass (g)
Mix 1: HL 3 1.5% RAS and 13.5% RAP	A	150.0	76.8	2983.5	150.0	72.0	2985.8	0.0	4.8	-2.3
	B	150.0	73.0	2988.6	150.0	72.0	2991.2	0.0	1.0	-2.6
	C	150.0	73.6	3001.2	150.0	72.0	3003.8	0.0	1.6	-2.6
Mix 2: SP 19 6% RAS	2A	150.0	74.0	2995.1	149.0	73.0	2995.7	1.0	1.0	-0.6
	2B	150.0	74.2	3003.6	149.0	74.0	3007.2	1.0	0.2	-3.6
	2C	150.0	73.8	2983.0	150.0	73.0	2986.5	0.0	0.8	-3.5
Mix 3: SP 19 3% RAS and 25%RAP	3A	150.0	73.8	2996.6	149.0	72.0	3000.1	1.0	1.8	-3.5
	3B	150.0	73.5	2995.5	149.0	72.0	2998.9	1.0	1.5	-3.4
	3C	150.0	73.6	2994.0	149.0	73.0	2997.2	1.0	0.6	-3.2
Mix 4: SP12.5 FC1 3% RAS and 12%RAP	4A	150.0	72.5	2994.5	149.0	71.0	2997.6	1.0	1.5	-3.1
	4B	150.0	71.9	2991.2	149.0	71.0	2994.3	1.0	0.9	-3.1
	4C	150.0	71.6	2995.8	149.0	71.0	2998.7	1.0	0.6	-2.9
Mix 5: SP12.5 FC2 6% RAS	5A	150.0	72.3	2993.6	149.0	71.0	2998.1	1.0	1.3	-4.5
	5B	150.0	74.2	2995.2	149.0	72.0	2997.6	1.0	2.2	-2.4
	5C	150.0	74.5	3002.5	149.0	73.0	3004.8	1.0	1.5	-2.3
Mix 6: SP12.5 FC2 3% RAS and 12% RAP	6A	150.0	72.0	3045.3	150.0	73.0	3049.8	0.0	-1.0	-4.5
	6B	150.0	72.0	3003.4	150.0	72.0	3007.9	0.0	0.0	-4.4
	6C	150.0	72.2	2995.9	149.0	71.0	2999.8	1.0	1.2	-3.9

Table 4-7: After Second Year Physical Properties Statistical Analysis

	Initial Diameter (mm)	***2nd Cycle Diameter (mm)***	***Initial Height (mm)***	***2nd Cycle Height (mm)***	***Initial Mass (g)***	***2nd Cycle Mass (g)***
Mean	150	149.3	73.3	72.1	2997.7	3000.8
Variance	0	0.24	1.6	0.8	175.2	186.0
Observations	18	18	18	18	18	18
Degree of Freedom (df)	17	17	17	17	17	17
F	**0**		**2.02**		**0.94**	
P(F<=f) one-tail	**0**		**0.08**		**0.45**	
F Critical one-tail	**0.44**		**2.27**		**0.44**	

The Surface texture test performed on the asphalt slabs indicated that the surface was still in good condition. However, when compared to the initial surface texture condition, there was a decrease in mean texture depth as shown in Table 4-8. This demonstrates the loss of coarse aggregates during the freeze-thaw cycling with time. Unlike the first freeze-thaw cycle, there was a statistically significant change in texture depth as demonstrated by Table 4-9. The $F_{calculated} > F_{Critical}$ and P-Value > 0.1 indicates weak evidence of consistence with the initial texture characteristics hence as excepted with time pavement surface texture tends to change or deteriorate. The slabs were still in good condition by the end of the second year set of cycles.

Table 4-8: After Second Year (Second Set of Freeze-Thaw Cycles) Surface Texture

Mix Description	Slab Number	Initial MTD (mm)	2nd Cycle MTD (mm)	Difference
Mix 1: HL 3 1.5% RAS and 13.5% RAP	A	0.05	0.03	0.01
	B	0.05	0.03	0.02
	C	0.04	0.04	0.01
Mix 2: SP 19 6% RAS	2A	0.05	0.04	0.01
	2B	0.04	0.03	0.01
	2C	0.04	0.03	0.01
Mix 3: SP 19 3% RAS and 25%RAP	3A	0.04	0.03	0.01
	3B	0.04	0.03	0.01
	3C	0.04	0.03	0.01
Mix 4: SP12.5 FC1 3% RAS and 12%RAP	4A	0.04	0.03	0.00
	4B	0.04	0.03	0.01
	4C	0.04	0.03	0.01
Mix 5: SP12.5 FC2 6% RAS	5A	0.06	0.07	-0.01
	5B	0.05	0.05	0.00
	5C	0.07	0.07	0.00
Mix 6: SP12.5 FC2 3% RAS and 12% RAP	6A	0.06	0.07	-0.01
	6B	0.08	0.05	0.03
	6C	0.05	0.05	0.00

Table 4-9: Surface Texture Statistical Analysis (After Second Year)

Description	***Initial MTD (mm)***	***2nd Cycle MTD (mm)***
Mean	0.05	0.04
Variance	0.00	0.00
Observations	18	18
Degree of Freedom (df)	17	17
F	**0.66**	
P(F<=f) one-tail	0.20	
F Critical one-tail	**0.44**	

4.2.1.4 ***Physical Properties Summary***

Generally; there was a significant change in the physical properties of the asphalt slabs after the two freeze-thaw cycles. The surface of the pavement slab exhibited no significant visible distresses or aggregate loss and were observed to in good condition by the end of the analysis period. All the HMA slabs were observed to increase in mass, and decrease in height without significant change in diameter. The change in mass and height would be due to the expansion and contraction of the slabs during the freeze-thaw cycles. However, all the slabs held up well during the analysis period with no visible signs of breakage. The results illustrated promising performance for use of HMA mixtures containing RAS under Ontario climatic changes. There was no statistical difference between the RAS and/or RAP six mixes and the control mix.

4.2.2 Fiction Testing Using the British Pendulum

Friction testing using a British Pendulum Tester (BPT), which is dependent on the temperature of the pavement and rubber type when using natural rubber slider (TRL Slider), was used for testing. A temperature correlation is required to be performed on the BPN values attained during testing to normalize the BPN values at room temperature. Previous research indicated that friction measurements are greatly affected by temperature; whereby a higher temperature results in lower friction values. However, this influence is dependent on vehicle speed and pavement type [Lu et al 2006]. The British Pendulum can only be used at low slip speed reflecting low driving speed. Lu carried out a study on the effects of pavement temperature using the SN data measured by the Locked-Wheel Skid Tester [Luo 2003]. Basing on Kissoff's correlation equation relating BPN and Skid Number (SN), the skid number was calculated as shown in Equation 4-1 [Kissoff 1998].

$$SN = 0.862BPN - 9.69$$

Equation 4-1

Bazlamit et al further derived a similar equation to adjust the skid number at any temperature using Equation 4-2 [Bazlamit et al 2005].

$$\Delta SN_{T(K)} = 58.453 - 0.1994T(K)$$

Equation 4-2

Where; $\Delta SN_{T(K)}$ is number to be added to SN reading at T(K) = 293.15 (≈ 200C), and T(K) is temperature in Kelvin.

Both Equation 4-1 and Equation 4-2 were used to determine the SN at 20^0C which value was later compared with friction values given in Table 4-10 to determine the friction properties. A BPN rating given in Table 4-11 was also analyzed to correlate the values with performance of a vehicle braking with locked wheels on a wet pavement stopping from 50Kph [ICPI 2004].

Table 4-10: Criteria for Establishing Friction Properties on Pavement Surface [TAC 1997]

Category	Skid Number (SN)	Accident Prone	Action to be taken
1	< 30	Yes	General Maintenance/ Improvement programs required to increase safety
2	31 - 34	Yes	Maintain surveillance and take corrective action as required
3	34 or less	No	Maintain surveillance and take corrective action as required
4	34 - 40	No	Maintain surveillance and take corrective action as required
5	> 40	-	No action is required

Table 4-11: BPN Rating [ICPI 2004]

Category	British Pendulum Number (BPN)	Action to be taken
1	45 - 55	Satisfactory surface in only favorable conditions (weather and vehicle)
2	55 - 65	Generally acceptable skid resistance in all but most severe weather condition
3	> 65	Good to excellent skid resistance in all conditions

4.2.2.1 *Initial Friction Evaluation*

The British Pendulum Number (BPN) values obtained from the test were in the range of 43 – 56; hence, most of the slabs were within the minimum acceptable skid resistance value of 45 for all types of pavements as given in Table 4-11. The initial British Pendulum Number/Skid Resistance Numbers of the slab are given in Table 4-12. The pavement surfaces were observed to be in satisfactory condition to provide safety condition during wet conditions.

Table 4-12: Initial British Pendulum Number of the Asphalt pavement Slabs

Slab Number	Temperature of Water on Pavement	Number of Tests					Mean BPN	Corrected BPN	SN (20^0C)	Standard Deviation
		1	2	3	4	5				
Mix 1: HL 3 A	21.0	42	45	44	41	42	43	43	27	1.6
Mix 1: HL 3 B	21.0	45	47	47	48	45	46	47	31	1.3
Mix 1: HL 3 C	20.4	58	59	59	59	59	59	59	41	0.4
Mix 2A	20.4	51	52	50	52	49	51	51	34	1.3
Mix 2B	20.4	47	47	47	47	47	47	47	31	0.0
Mix 2C	20.4	46	45	45	44	45	45	45	29	0.7
Mix 3A	20.8	45	45	44	44	43	44	44	29	0.8
Mix 3B	20.4	48	50	49	46	46	48	48	32	1.8
Mix 3C	20.4	49	47	45	43	42	45	45	29	2.9
Mix 4A	20.8	48	45	43	42	41	44	44	28	2.8
Mix 4B	20.4	48	47	46	45	45	46	46	30	1.3
Mix 4C	21.2	50	50	50	45	56	50	51	34	3.9
Mix 5A	20.8	52	54	50	53	53	52	53	36	1.5
Mix 5B	20.0	55	52.5	50	50	49	51	51	35	2.4
Mix 5C	19.4	56	54	53	52	52	53	53	36	1.7
Mix 6A	20.2	49	48	48	47	46	48	48	31	1.1
Mix 6B	19.4	50	49	47	46	46	48	47	31	1.8
Mix 6C	20.4	60	58	56.5	54	53	56	56	39	2.9

4.2.2.2 *After One Year (First Set of Freeze-Thaw Cycles): Friction*

An increase in BPN was generally observed for all the slabs and mix type after the first set of freeze-thaw cycles as given in Table 4-13. Mix 3: SP 19 3% RAS, 25% RAP and Mix 1 were observed to exhibit the highest rate of increase in friction resistance properties compared to other mixes. Mix 5: SP12.5 FC2 6% RAS and Mix 6: SP12.5 FC2 3% RAS, 12% RAP. However, the slabs were consistent in their smooth nature as previously noted. The skid number was observed to be between

34 to 40; indicating that HMA mixes containing RAS can provide skid resistance pavement surfaces that are safe (surfaces are not accident prone as given in Table 4-10); however, monitoring is advisable so that corrective maintenance can be provided when required.

Table 4-13: After One Year (First Freeze-Thaw Cycle) Friction Testing

Slab Number	Temperature of Water on Pavement	Number of Tests					Mean BPN	Corrected BPN	SN (20^0C)	Standard Deviation
		1	2	3	4	5				
HL 3 A	20.6	55	60	61	60	52	58	58	40	3.9
HL 3 B	20.0	58	58	55	55	55	56	56	39	1.6
HL 3 C	20.0	58	56	56	56	54	56	56	38	1.4
Mix 2A	20.8	52	50	56	62	47	53	54	37	5.8
Mix 2B	19.6	55	54	53	56	51	54	54	37	1.9
Mix 2C	21.0	53	52	51	51	51	52	52	35	1.0
Mix 3A	20.8	52	52	51	50	49	51	51	34	1.3
Mix 3B	21.0	55	52	55	53	52	53	54	37	1.5
Mix 3C	21.0	54	52	51	52	52	52	52	35	1.1
Mix 4A	21.8	59	50	48	48	46	50	51	34	5.1
Mix 4B	21.8	53	50	50	49	50	50	51	34	1.5
Mix 4C	20.8	56	53	53	51	51	53	53	36	2.0
Mix 5A	21.0	57	54	52	51	52	53	53	36	2.4
Mix 5B	21.4	57	54	55	54	53	55	55	38	1.5
Mix 5C	20.8	50	52	50	48	48	50	50	33	1.7
Mix 6A	17.8	55	55	52	51	50	53	52	35	2.3
Mix 6B	19.6	54	51	54	50	50	52	52	35	2.0
Mix 6C	21.2	53	48	47	50	46	49	49	33	2.8

A comparison between initial friction values and after the first year cycles of freeze-thaw conditions indicated a huge change in the friction properties. The asphalt slab surface was observed to have increased in friction properties hence safety as given in Table 4-14. The F-test performed on the initial and first set of freeze-thaw set of cycles on the samples indicated statistical significance as given in Table 4-15. The $F_{Calculated} > F_{Critical}$ and P-Value indicate strong evidence of an overall change in frictional resistance properties among the mixes (samples). However, the British Pendulum Number (BPN) was still greater than 45 indicating that the pavements would be safe to the road users.

Table 4-14: After One Year (First Set of Freeze-Thaw Cycles) Friction Result Comparison

Mix Description	Slab Number	BPN (Initial)	Standard Deviation (Initial)	BPN (1st Cycle)	Standard Deviation (1st Cycle)	BPN (Difference)	Standard Deviation (Difference)
Mix 1: HL 3 1.5% RAS and 13.5% RAP	A	43.0	1.6	57.8	3.9	-14.8	-2.3
	B	46.6	1.3	56.2	1.6	-9.5	-0.3
	C	58.9	0.4	55.7	1.4	3.2	-1.0
Mix 2: SP 19 6% RAS	2A	50.9	1.3	53.6	5.8	-2.7	-4.5
	2B	47.1	0.0	53.7	1.9	-6.6	-1.9
	2C	45.1	0.7	51.8	1.0	-6.7	-0.3
Mix 3: SP 19 3% RAS and 25%RAP	3A	44.4	0.8	51.0	1.3	-6.6	-0.5
	3B	47.9	1.8	53.7	1.5	-5.8	0.3
	3C	45.3	2.9	52.4	1.1	-7.1	1.7
Mix 4: SP12.5 FC1 3% RAS and 12%RAP	4A	44.0	2.8	50.7	5.1	-6.7	-2.3
	4B	46.3	1.3	50.9	1.5	-4.6	-0.2
	4C	50.5	3.9	53.0	2.0	-2.5	1.8
Mix 5: SP12.5 FC2 6% RAS	5A	52.6	1.5	53.5	2.4	-0.9	-0.9
	5B	51.3	2.4	55.0	1.5	-3.7	0.9
	5C	53.2	1.7	49.8	1.7	3.4	0.0
Mix 6: SP12.5 FC2 3% RAS and 12% RAP	6A	47.7	1.1	52.0	2.3	-4.3	-1.2
	6B	47.5	1.8	51.7	2.0	-4.2	-0.2
	6C	56.4	2.9	49.1	2.8	7.3	0.1

Table 4-15: After One Year Friction Statistical Analysis

Description	*BPN (Initial)*	*BPN (1st Cycle)*
Mean	48.82	52.86
Variance	19.31	5.25
Observations	18	18
Degree of freedom (df)	17	17
F Calculated	**3.68**	
P(F<=f) one-tail	**0.01**	
F Critical one-tail	**2.27**	

4.2.2.3 ***After Second Year (Second Set of Freeze-Thaw Cycles): Friction***

There was a slight decrease in friction properties when compared to the initial friction resistance properties for all slabs as given in Table 4-16 and Table 4-17. The skid number for all the slabs was observed to be below 30 indicating the need for maintenance programs in order to improve on road safety measures. This was further supported by BPN number as they were below 45 demonstrating satisfactory conditions only in favorable conditions such as dry weather. Mix 5: SP12.5 FC2 6% RAS had the highest rate of decrease in friction resistance properties followed by Mix 6: SP12.5 FC2 3% RAS, 12% RAP both of which are surface mixes. Mix 3: SP19 3% RAS and 25% RAP had the slowest rate in loss of friction resistance properties. The final asphalt slab properties are shown in Figure 4-2.

Table 4-16: After Second Year (Second Set of Freeze-Thaw Cycles) Friction Testing

Slab Number	Temperature of Water on Pavement	Number of Tests					Mean BPN	Corrected BPN	SN (20^0C)	Standard Deviation
		1	2	3	4	5				
HL 3 A	23.8	44	42	41	40	38	41	42	26	2.2
HL 3 B	24.0	43	43	41	40	40	41	42	27	1.5
HL 3 C	23.6	42	41	40	40	39	40	41	26	1.1
Mix 2A	23.8	41	40	38	38	36	39	39	24	1.9
Mix 2B	24.0	43	43	39	38	37	40	41	26	2.8
Mix 2C	24.4	41	39	38	36	36	38	39	24	2.1
Mix 3A	24.4	42	39	37	37	35	38	39	24	2.6
Mix 3B	24.2	41	39	39	39	40	40	40	25	0.9
Mix 3C	24.0	48	44	42	42	43	44	45	29	2.5
Mix 4A	23.8	42	40	37	37	37	39	39	24	2.3
Mix 4B	23.8	42	41	40	38	38	40	41	25	1.8
Mix 4C	24.0	38	37	38	36	35	37	38	23	1.3
Mix 5A	24.4	45	42	41	37	40	41	42	26	2.9
Mix 5B	23.4	41	39	36	36	34	37	38	23	2.8
Mix 5C	24.4	42	38	40	37	36	39	40	24	2.4
Mix 6A	24.4	41	39	38	37	36	38	39	24	1.9
Mix 6B	24.2	40	38	36	36	35	37	38	23	2.0
Mix 6C	24.2	44	42	40	40	40	41	42	27	1.8

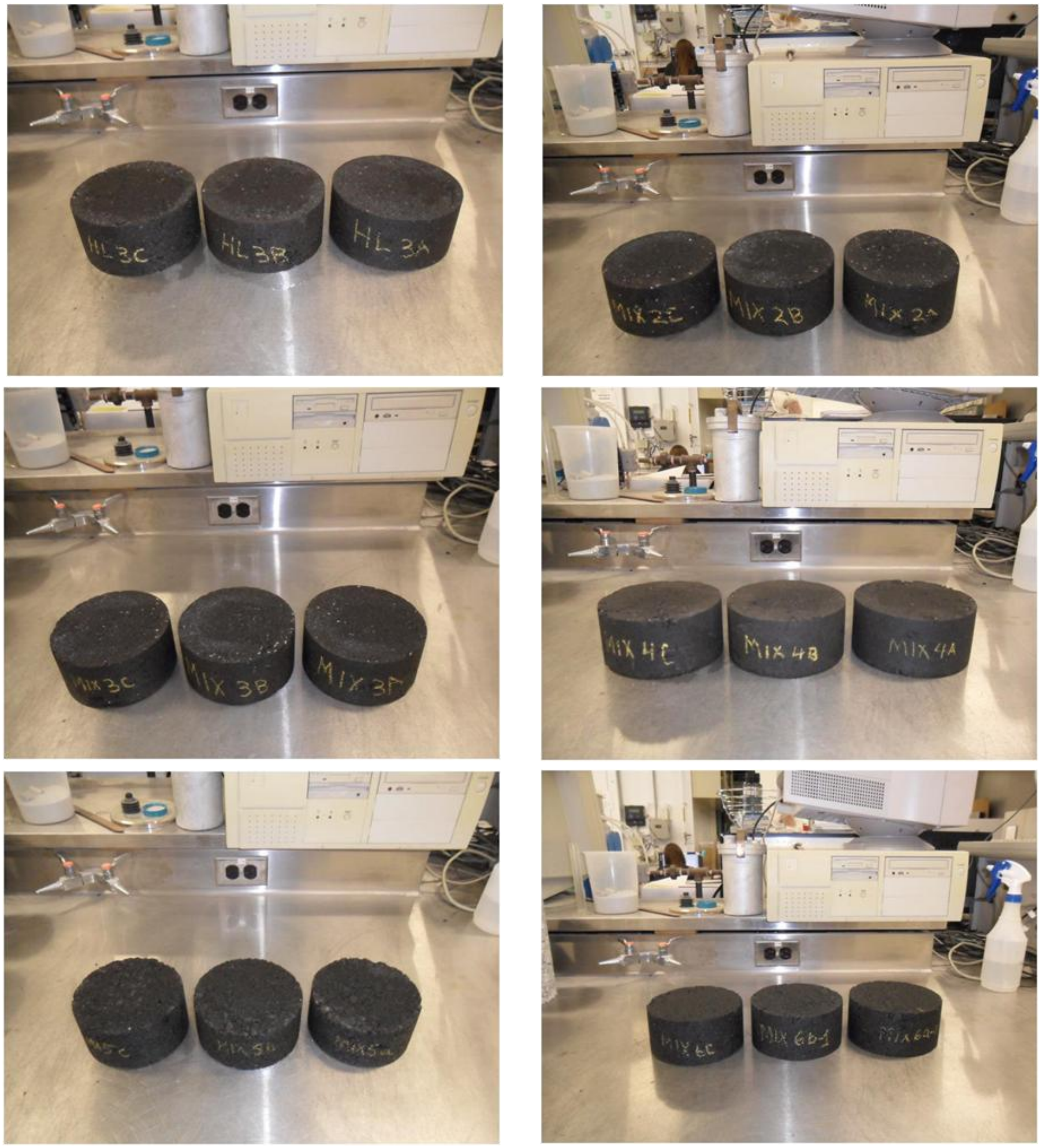


Figure 4-2: Final Asphalt Slab Properties

The F-test carried on the BPN results indicated strong statistically significant evidence of variation in friction resistance properties from the initial properties of the asphalt slabs which is supported by the P-Value > 0.001 as given in Table 4-18. The $F_{Calculated} > F_{Critical}$ illustrates the frictional properties to have changed from the initial values hence they were not consistent with the original values which is expected to happen when a pavement is in service.

Table 4-17: After Second Year (Second Set of Freeze-Thaw Cycles) Friction Result Comparison

Mix Description	Slab Number	BPN (Initial)	Standard Deviation (Initial)	BPN (2nd Cycle)	Standard Deviation (2nd Cycle)	BPN (Difference)	Standard Deviation (Difference)
Mix 1: HL 3 1.5% RAS and 13.5% RAP	A	43.0	1.6	41.8	2.2	1.2	-0.6
	B	46.6	1.3	42.3	1.5	4.4	-0.2
	C	58.9	0.4	41.2	1.1	17.7	-0.7
Mix 2: SP 19 6% RAS	2A	50.9	1.3	39.4	1.9	11.5	-0.6
	2B	47.1	0.0	40.9	2.8	6.2	-2.8
	2C	45.1	0.7	38.9	2.1	6.2	-1.4
Mix 3: SP 19 3% RAS and 25%RAP	3A	44.4	0.8	38.9	2.6	5.5	-1.8
	3B	47.9	1.8	40.5	0.9	7.4	0.9
	3C	45.3	2.9	44.7	2.5	0.6	0.4
Mix 4: SP12.5 FC1 3% RAS and 12%RAP	4A	44.0	2.8	39.4	2.3	4.6	0.5
	4B	46.3	1.3	40.6	1.8	5.7	-0.5
	4C	50.5	3.9	37.6	1.3	12.9	2.6
Mix 5: SP12.5 FC2 6% RAS	5A	52.6	1.5	42.0	2.9	10.7	-1.4
	5B	51.3	2.4	37.9	2.8	13.4	-0.3
	5C	53.2	1.7	39.5	2.4	13.7	-0.7
Mix 6: SP12.5 FC2 3% RAS and 12% RAP	6A	47.7	1.1	39.1	1.9	8.5	-0.8
	6B	47.5	1.8	37.8	2.0	9.6	-0.2
	6C	56.4	2.9	42.1	1.8	14.3	1.1

Table 4-18: After Second Year Friction Statistical Analysis

	BPN (Initial)	*BPN (2nd Cycle)*
Mean	48.82	40.25
Variance	19.31	3.55
Observations	18	18
Degree of Freedom (df)	17	17
F Calculated	**5.44**	
P(F<=f) one-tail	**0.0005**	
F Critical one-tail	**2.27**	

4.2.2.4 ***Friction Properties Summary***

In general; friction properties were observed to change with time, initially after the asphalt slabs were subjected to first cycle of freeze-thaw the friction properties increased greatly. However; after the second cycle, it was observed that the friction properties decreased even further from the initial BPN as given in Table 4-19. Mix 3: SP19 3%RAS and 25% RAP was observed to perform best over time. It had the highest increase in BPN as well as the slowest rate of friction resistance loss followed by Mix 1.

Table 4-19: Friction Properties for Asphalt Mixes

Mix Description	Slab Number	BPN (Initial)	BPN (1st Cycle)	BPN (2nd Cycle)
Mix 1: HL 3 1.5% RAS and 13.5% RAP	A	43	58	42
	B	47	56	42
	C	59	56	41
Mix 2: SP 19 6% RAS	2A	51	54	39
	2B	47	54	41
	2C	45	52	39
Mix 3: SP 19 3% RAS and 25%RAP	3A	44	51	39
	3B	48	54	40
	3C	45	52	45
Mix 4: SP12.5 FC1 3% RAS and 12%RAP	4A	44	51	39
	4B	46	51	41
	4C	51	53	38
Mix 5: SP12.5 FC2 6% RAS	5A	53	53	42
	5B	51	55	38
	5C	53	50	40
Mix 6: SP12.5 FC2 3% RAS and 12% RAP	6A	48	52	39
	6B	47	52	38
	6C	56	49	42

4.3 Field Pavement Condition Evaluation

To validate the performance of asphalt pavements with RAS, two test locations were constructed; one at the CPATT Test Track and three residential streets in the Town of Markham, Ontario Canada. All the four test sections are performing well and no significant visual signs of surface distresses were noticed. A field survey on both test sections, under study at the CPATT Test Track and in the Town of Markham, was carried out in June 2011 following MTO guidelines and evaluation form found in Appendix A. Pavement surface distress survey was carried out on the residential streets while an additional test (PFWD) was performed on the CPATT Test Track. Both sections were observed to be in excellent condition to support the designed traffic loading as well as withstand the Canadian climatic changes.

4.3.1 Previous Field Evaluation

4.3.1.1 *Pavement Surface Distress*

Pavement evaluations previously carried out in June 2010 under Phase 2 indicated that Ida Street had slight noticeable longitudinal and transverse cracking (3mm to 5mm in width), segregation, aggregate loss, and small pop outs of approximately 10mm to 15mm which did not contain any RAS content. Whereas, the CPATT Test Track, Paul Street and Vintage Lane, and Thornhill Summit Drive were still in excellent conditions to support the designed for load without noticeable distresses. The overall evaluation indicated that all the test sections were still performing very well as shown in Figure 4-3 and Figure 4-10 [UL-Islam 2010].

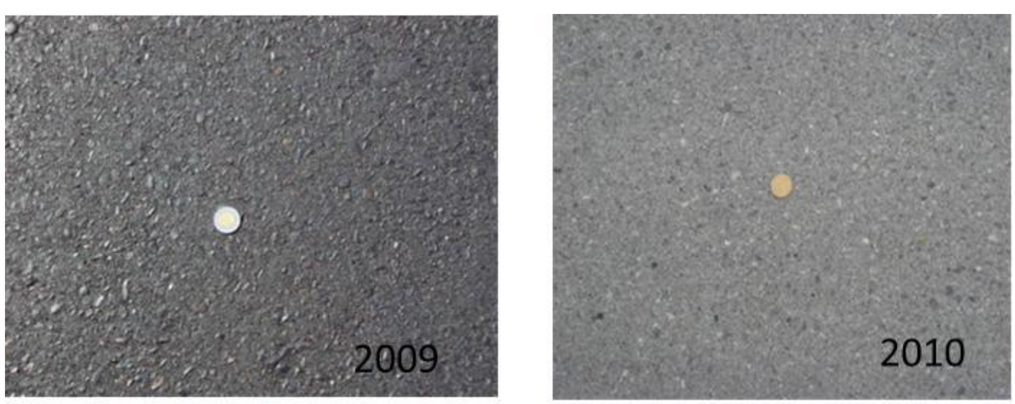


Figure 4-3: CPATT Test Track Pavement Surface Characteristics [UL-Islam 2010]

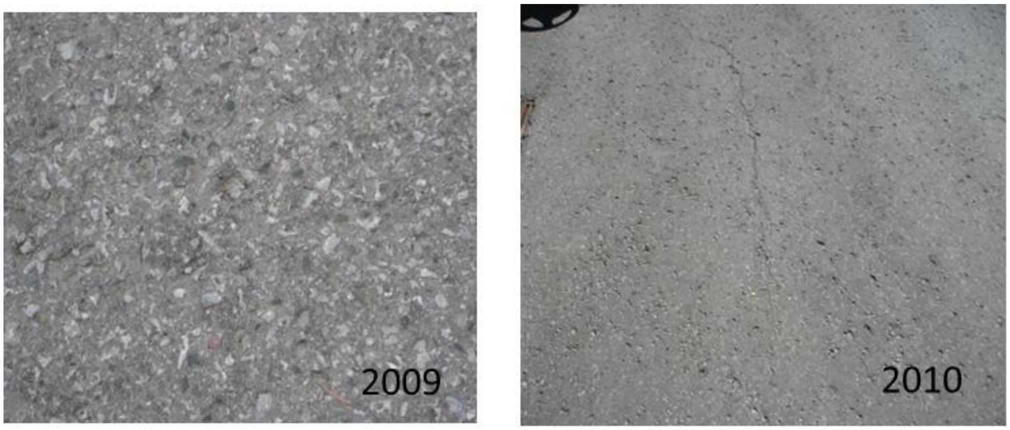


Figure 4-4: Ida Street Pavement Surface Characteristics [UL-Islam 2010]

Figure 4-5: Paul St and Vintage Lane Pavement Surface Characteristics [UL-Islam 2010]

Figure 4-6: Thornhill Summit Drive Pavement Surface Characteristics [UL-Islam 2010]

4.3.1.2 ***Deflection Measurement at CPATT Test Track***

Deflection measurements were performed at the CPATT Test Track in June 2010 using CPATT's Portable Falling Weight Deflectometer (PFWD). It was observed that the Right Wheel Path had higher deflection values than the Left Wheel Path as well as more was noticed in the North East lane than southwest lane as shown in Figure 4-7 and Figure 4-8. This could be explained by the fact that the northeast lane supports trucks carrying a heavier load increasing the load on the pavement compared to the South West lane which carries empty truck from the landfill. However, higher deterioration rate in the Right Wheel Path than the Left Wheel Path suggests that it could be due to lack of a paved road or no shoulder, and unstable road edge leading to lateral movement into the unpaved side road edge.

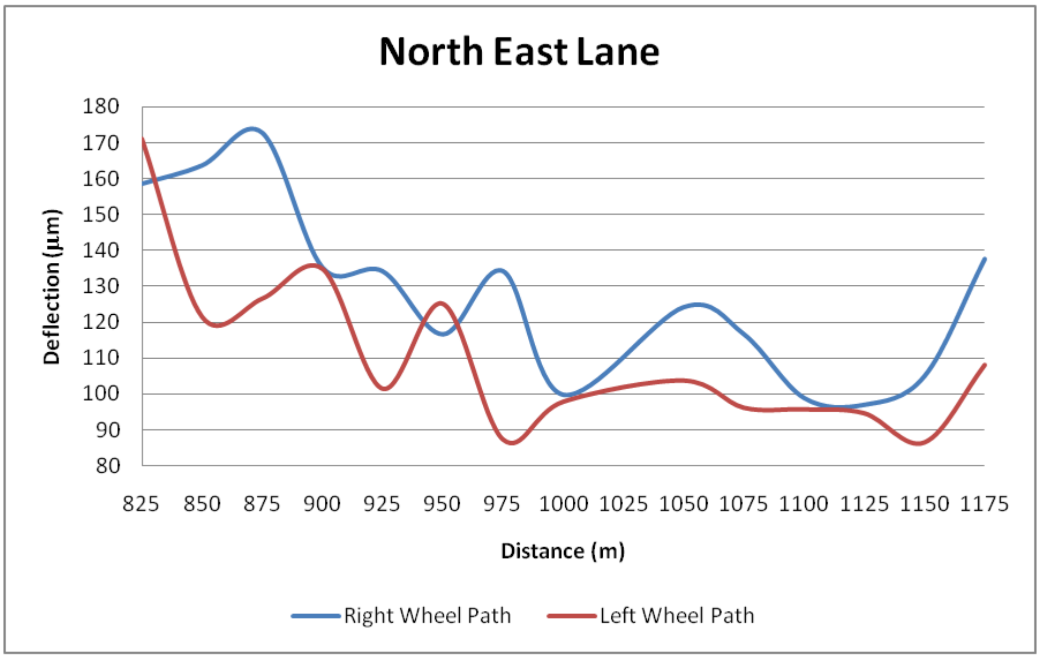


Figure 4-7: Rate of Deterioration in the Wheel Paths (North East Lane)

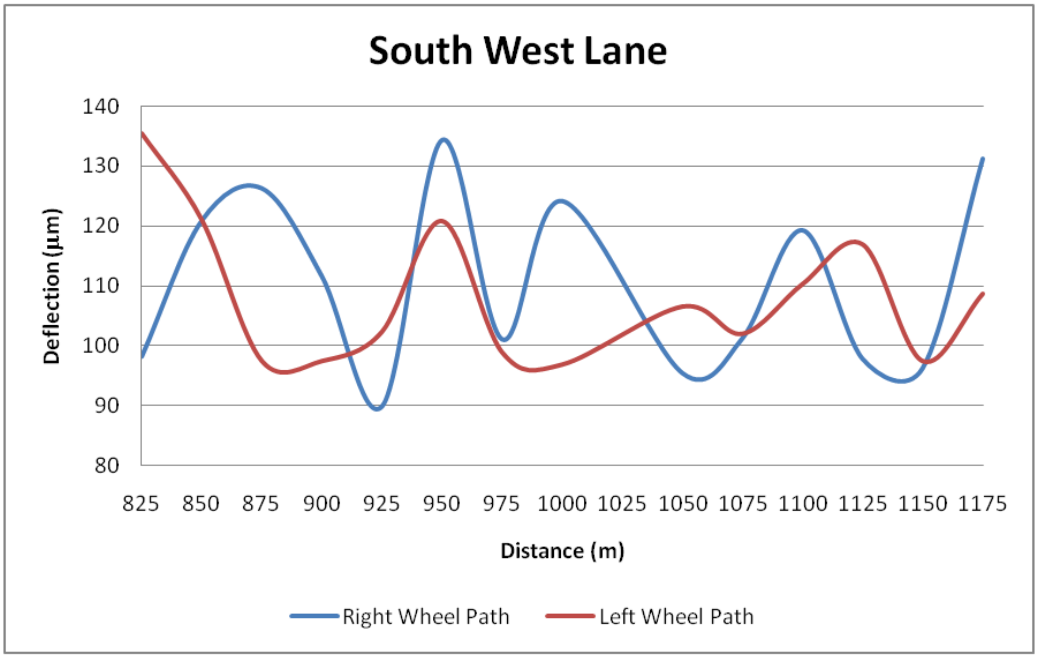


Figure 4-8: Rate of Deterioration in the Wheel Paths (South West Lane)

Statistically; variation was observed between deflection measurements in different lane directions than within same lane direction as given in Table 4-20 and the $F_{Calculated} > F_{Critical}$. This is explained by the fact that one lane carried a load double the other lane. The P-Value of the analysis in both directions indicated strong evidence of differences between the different lane directions, different wheel paths but shows consistency in the data.

Table 4-20: Analysis of Variance (ANOVA) for Deflection Measurements

Source of Variation	***Sum of Squares (SS)***	***Degree of Freedom (df)***	***Mean Squares (MS)***	$F_{Calculated}$	***P-value***	$F_{critical}$
Within Lane Deflection	9584	13	737	2.9	0.01	2.0
Between Lane Directions	3595	3	1198	4.7	0.01	2.8
Error	9989	39	256			
Total	23168	55				

Overall the CPATT Test Track was observed to be in very good condition after a year of construction supporting heavy traffic loading.

4.3.2 CPATT Test Track

4.3.2.1 *Pavement Surface Distress*

A distress survey was carried out on the RAS incorporated section on June 5th, 2011 to evaluate the overall pavement condition after having gone through two complete years of freeze-thaw cycles. Except for few deformations due to the construction machines and/or equipment in the facility, very slight frequent centre line cracking, and ravelling/course aggregate loss were observed especially in the wheel track path. Overall, the pavement was observed to be in excellent condition to support the heavy traffic loading without significant noticeable distresses as shown in Figure 4-15. It should be noted that the loaded lane is in the southbound (SB) direction while the unloaded lane is in the northbound (NB) direction. The observed distresses are shown in Figure 4-9 to Figure 4-14. Ravelling and coarse aggregate loss are shown in Figure 4-9, while construction equipment damage and initiation of pothole are shown in Figure 4-10 in the southbound. Distresses in the northbound include initiation of pothole shown in Figure 4-11, ravelling shown in Figure 4-12, and aggregate loss shown in Figure 4-13. Deterioration at the beginning of the RAS section was observed as shown in Figure 4-14.

Figure 4-9: SB – Ravelling and Course Aggregate Loss in Wheel Track Path

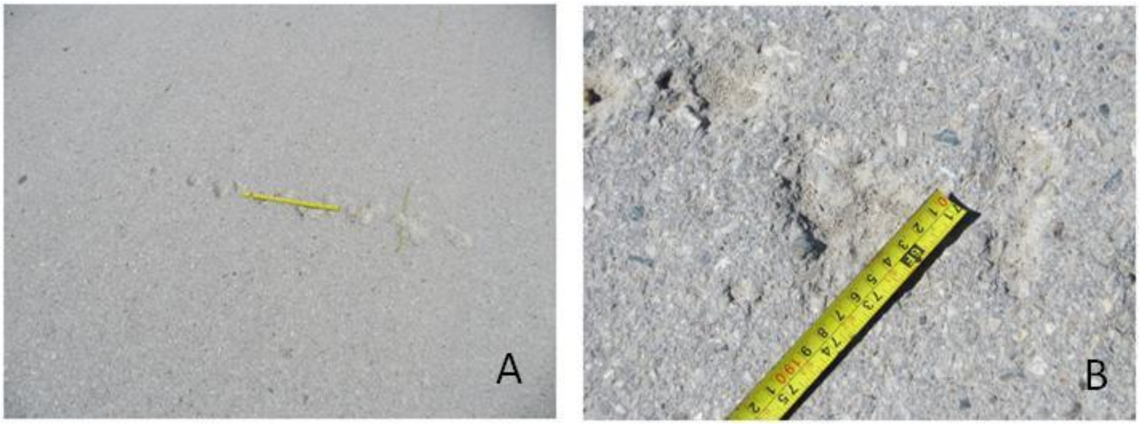


Figure 4-10: SB – Construction Equipment Damage (A) and Initiation of Pothole (B)

Figure 4-11: NB – Initiation of Potholes

Figure 4-12: NB – Ravelling in the Wheel Track Path

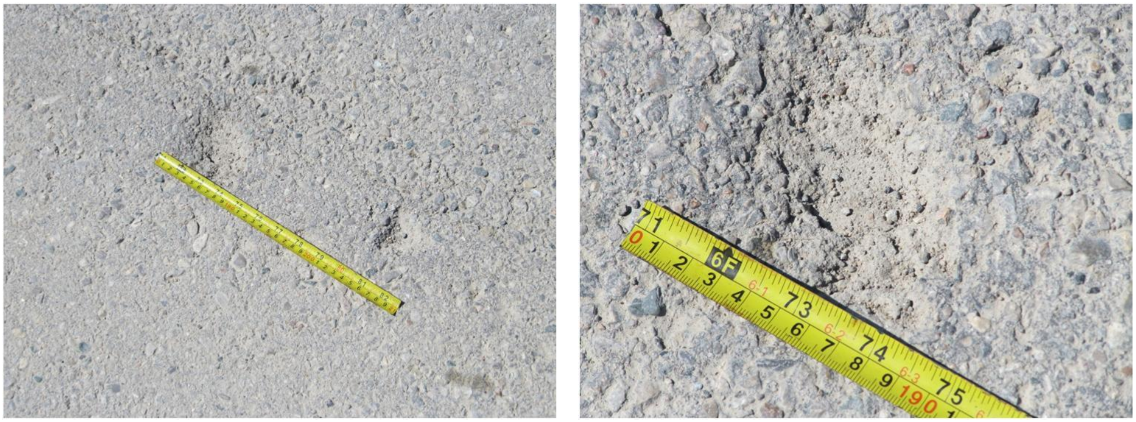


Figure 4-13: NB – Moderate Aggregate Loss resulting into Potholes

Figure 4-14: Failure at the beginning of the RAS Section

Figure 4-15: CPATT Test Track Comparison of Pavement Surface Characteristics

4.3.2.2 ***Deflection Measurement using the Pavement Falling Weight Deflectometer (PFWD)***

A non-destructive structural test was carried out using the PFWD to obtain the deflection measurements in both the right wheel path and left wheel path reflecting the pavement condition with time. The deflection testing was carried out every 100m intervals in each wheel path and was staggered at 50m as shown in Figure 4-16. The values were compared with the previous deflection measurements to determine the rate of deterioration with time. The deflection values were observed to be higher in the unloaded lane compared to the loaded lane as shown in Figure 4-17. This can be explained from the settlement due to compaction by the repeated traffic loading. The loaded lane seems to have completely settled onto the subgrade or it was fully compacted by the traffic loading it carries whereas the unloaded/return lane was still settling or it is more flexible.

However, at the end of the RAS section deflection was higher in the loaded lane compared to the unloaded lane as expected. The landfill entrance is located at chainage 1+100m; this explains the differences in deflection as illustrated in Figure 4-17.

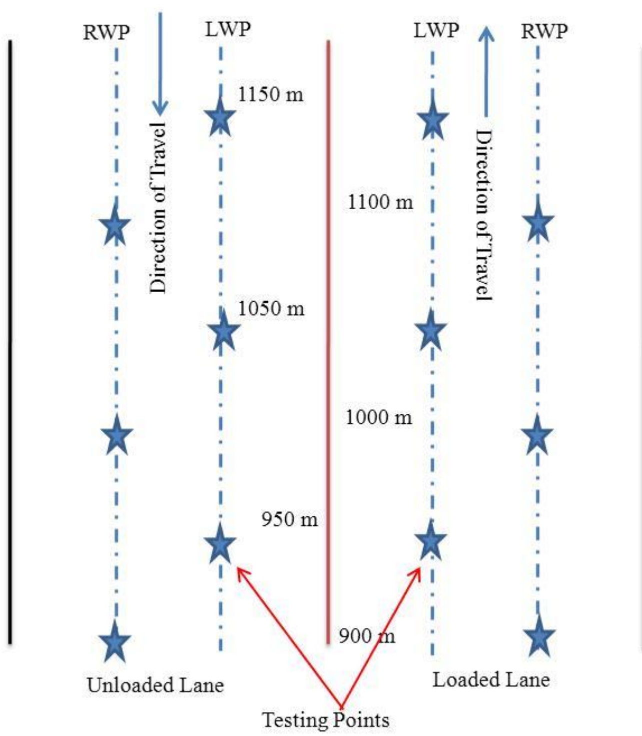


Figure 4-16: Illustration of Deflection Testing at CPATT Test Track

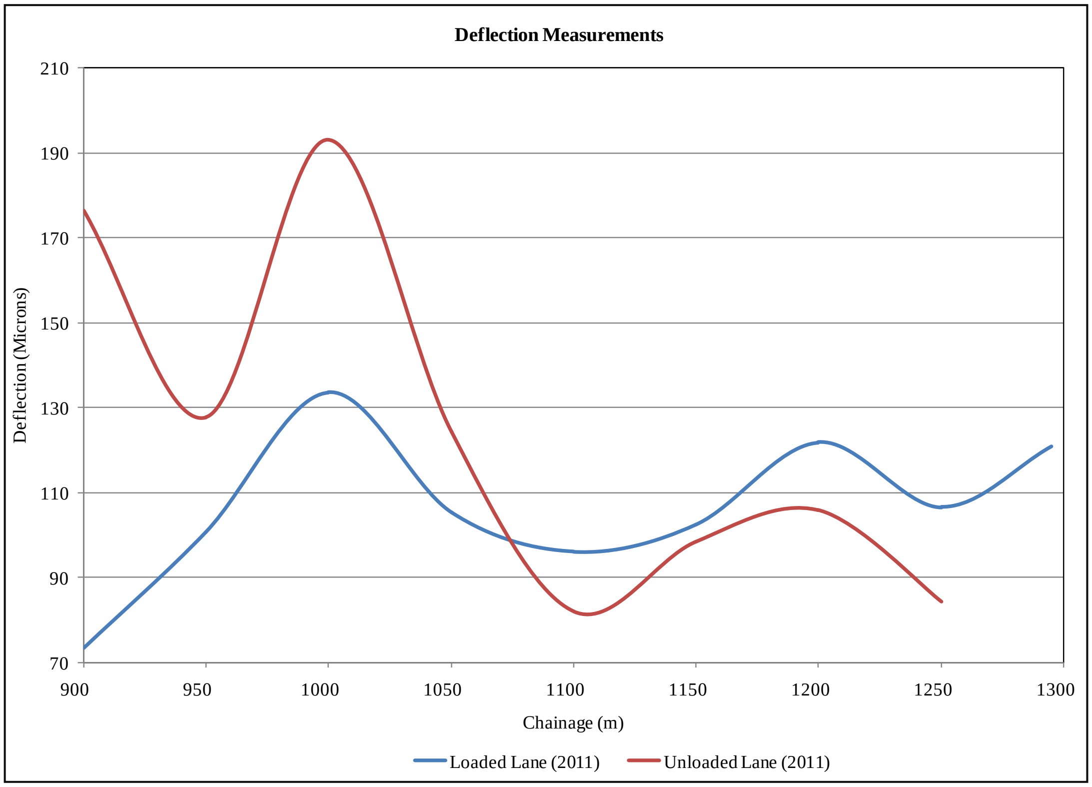


Figure 4-17: Deflection Measurements at the CPATT Test Track

4.3.2.3 ***Comparison of the Deflection Measurements at CPATT Test Track***

A statistical analysis was carried out on the deflection measurements performed in 2010 and 2011 to establish the variation in pavement performance in the two years since it was constructed and hence determines the rate of deterioration as shown in Figure 4-22. It was observed that the unloaded/return lane exhibited higher surface deflection compared to the loaded lane in both years of testing. However, at chainage 1+000 (1000m), higher increases in deflection were observed in both lanes from 2010 to 2011 hence there is a higher rate of deterioration in this particular location compared to the entire section. This could be explained based on the geometry of the road as it is a blind-spot therefore the vehicles slow down at this location compared to other parts of the section. With static motion the vehicles move at a much slower pace or sometimes come to complete braking position increasing the impact/grip of vehicle with the pavement surface.

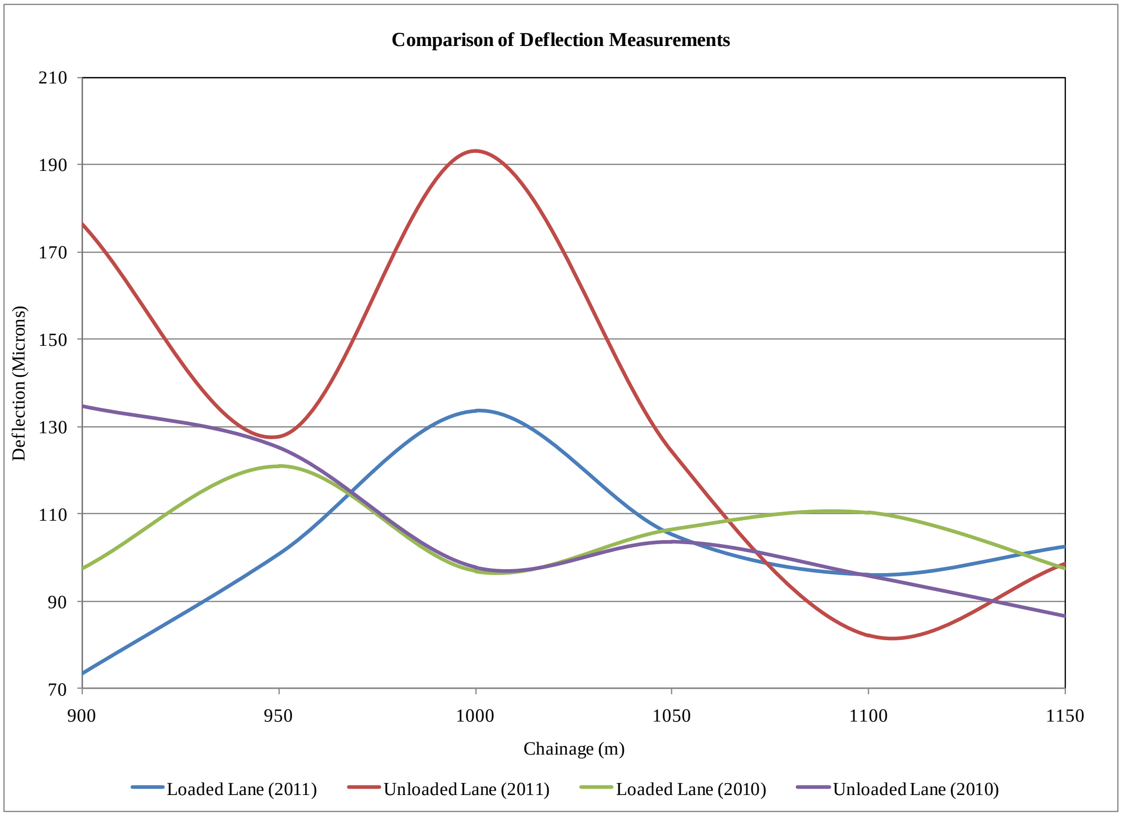


Figure 4-18: Comparison of Deflection Measurements at CPATT Test Track

However; based on the Analysis of Variance (ANOVA), it was observed that there was strong statistical evidence of consistency between the two sets of data and there was no significant differences in the surface deflection measurements between the two years. The pavement was still in very good condition to support the designed traffic loading as given in Table 4-21. Also it should be noted that at CPATT Test Track all loading is truck and in southbound lane, it explains very heavy loading. In both cases $F_{Critical} > F_{Calculated}$ and P-Value > 0.1indicate the pavement surface deflection being consistent with previous surface deflection.

Table 4-21: ANOVA – Comparison of Surface Deflection at CPATT Test Track

ANOVA: CPATT Test Track Loaded Lane						
Source of Variation	***Sum of Square***	***degree of freedom***	***Mean Square***	$F_{calculated}$	***P-value***	$F_{critical}$
Between Groups	26.7	1	26.7	0.12	0.74	4.96
Within Groups	2318.3	10	231.8			
Total	2345.1	11				
ANOVA: CPATT Test Track Unloaded Lane						
Source of Variation	***Sum of Square***	***degree of freedom***	***Mean Square***	$F_{calculated}$	***P-value***	$F_{critical}$
Between Groups	2093.4	1	2093.4	1.88	0.20	4.96
Within Groups	11128.3	10	1112.8			
Total	13221.7	11				

Therefore; the RAS pavement section was still intact and in good condition and performing very well to support the load after two years in-service recognizing it is exposed to very heavy traffic loading.

4.3.2.4 *Friction Testing at CPATT Test Track*

A friction test was carried out at CPATT Test Track using the British Pendulum to assess the safety properties of the pavement surface as illustrated by Figure 4-19. The test was carried out every 100m in the left wheel path while every 300m it was carried out in both the right wheel path and left wheel path as given in Table 4-22. British Pendulum Number (BPN) or Skid Resistance Value (SRV) is the analysis of the friction properties on the pavement which can be correlated with Skid Number to determine the safety of the pavement surface.

The BPN obtained from the test indicated that the RAS section was still in good safe condition to support the traffic loading it carries with BPN > 45, which was acceptable for heavy travelled roads as supported in the guidelines as well as SN > 40 [TAC 1997].

An estimated average standard deviation of 2.1% was achieved indicating that the average BPN was closer to the individual BPN hence there was consistency in data collected and it is reliable to make a conclusion. Both lanes exhibit no significant difference in the BPN and have consistent friction resistance properties. Overall, the pavement was in good condition in all weather and vehicle conditions with safe frictional properties and relatively smooth surface.

Table 4-22: Friction Testing at CPATT Test Track

Chainage	Test Direction	RWP / LWP	Temperature of Water on Pavement (^{0}C)	Number of Tests					Mean BPN	Corrected BPN	SN 20^{0}C	Std Dev
				1	2	3	4	5				
0+900	Loaded	L	22.8	64	66	69	70	71	68	69	50	2.9
0+900	Loaded	R	30.4	72	74	75	75	75	74	78	58	1.3
0+950	Unloaded	L	27.4	60	63	64	65	64	63	66	47	1.9
0+950	Unloaded	R	27.2	61	64	64	64	66	64	66	47	1.8
1+000	Loaded	L	27.2	61	64	65	65	64	64	66	47	1.6
1+050	Unloaded	L	28.6	60	65	65	66	66	64	67	48	2.5
1+100	Loaded	L	24.4	46	48	45	45	45	46	47	31	1.3
1+150	Unloaded	L	26.4	68	70	70	70	69	69	72	52	0.9
1+200	Loaded	L	23.6	60	61	64	64	64	63	64	45	1.9
1+200	Loaded	R	26.0	66	70	71	70	70	69	72	52	1.9
1+250	Unloaded	L	25.2	58	56	59	60	60	59	60	42	1.7
1+250	Unloaded	R	25.4	60	67	69	73	69	68	70	50	4.8

Notes: RWP – Right Wheel Path and LWP – Left Wheel Path

Figure 4-19: CPATT Test Track – Friction Testing using British Pendulum

4.3.3 Town of Markham, Ontario Canada

A pavement distress survey was carried out on three residential streets on June 8th, 2011 that were constructed in 2007 and are also part of the research. Each year CPATT has performed evaluations on these sections. From visual survey, the pavements in all the streets were observed to be in good condition and were still intact without major distresses. The roughness evaluation using International

Roughness Index (IRI) on the residential streets were; 1.5mm/m for Ida Street, 0.1mm/m for Paul Street and Vintage Lane, and 0.8mm/m for Thornhill Summit Drive indicating that all the sections were smooth, comfortable and safe

4.3.3.1 ***Site 1: Ida Street***

The low volume residential street was observed to be in good structural condition and also exhibited comfortable condition (value – 7, on the Riding Condition Rating - RCR Scale) with moderate pop-outs of less than 15mm throughout the entire street. It also exhibited few moderate multiple longitudinal cracking, and transverse cracking with a width of 3mm and 2.5mm, respectively. Few moderate wheel track rutting (depth of 1.5mm) and aggregate loss/ravelling were observed on the pavement surface. These distresses ranged from aggregate pop-outs, longitudinal and transverse cracking, ravelling, rutting to construction damage as shown in Figure 4-20 to Figure 4-23. However, construction equipment damage was observed in some few sections of the street as shown in Figure 4-24.

Figure 4-20: Site 1 - Aggregate Pop-outs

Figure 4-21: Site 1 - Longitudinal and Transverse Cracking

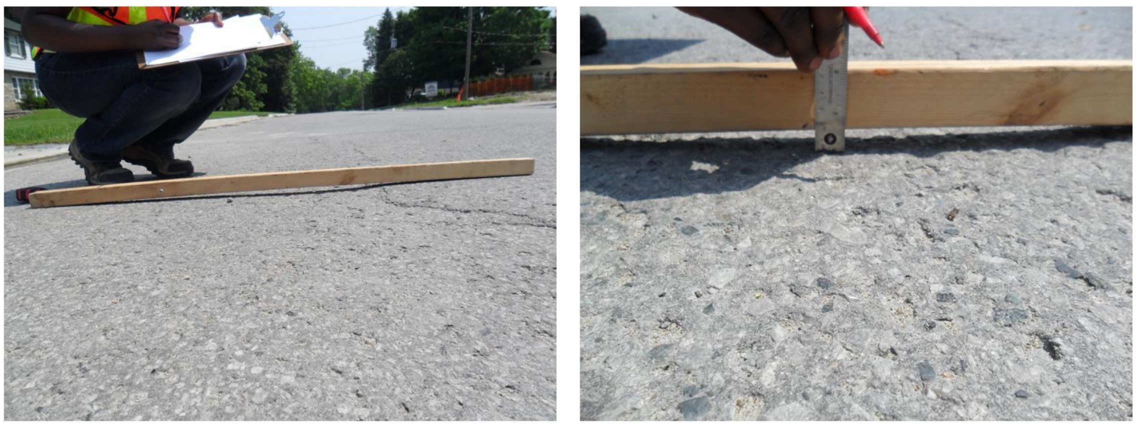


Figure 4-22: Site 1 - Wheel Track Rutting

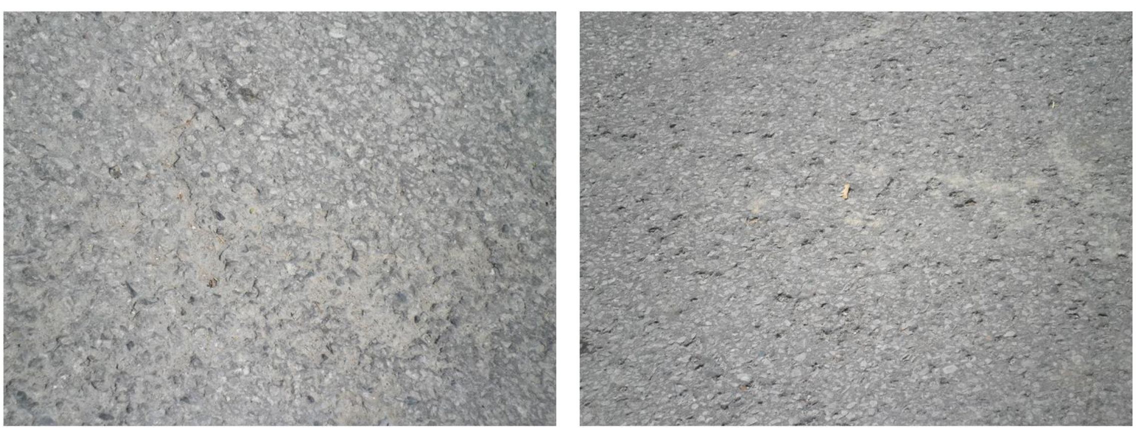


Figure 4-23: Site 1 – Ravelling and Aggregate Loss

Figure 4-24: Site 1 – Construction Equipment Damage

Figure 4-25: Site 1 - Comparison of Surface Characteristics

The pavement was observed to have slightly deteriorated as compared to the distress survey undertaken in 2010 as shown in Figure 4-25.

4.3.3.2 ***Site 2: Paul Street and Vintage Lane***

Paul Street and Vintage Lane was observed to be in better condition compared to Ida Street and Thornhill Summit Drive and exhibited a smooth comfortable (RCR value – 9) pavement surface when related to the Ride Condition Rating Scale (RCR) used by MTO. It had very slight intermittent transverse cracking (width of 2mm), slight intermittent multiple longitudinal cracking (width of 2mm) and ravelling/aggregate loss along the centreline as shown in Figure 4-26 to Figure 4-28. Few aggregate pop-outs were observed as well as very slight rutting (depth of 0.7mm).

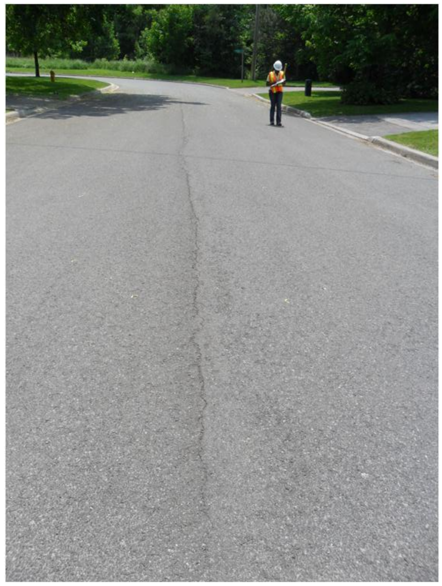



Figure 4-26: Site 2 – Longitudinal Cracking along the Centreline

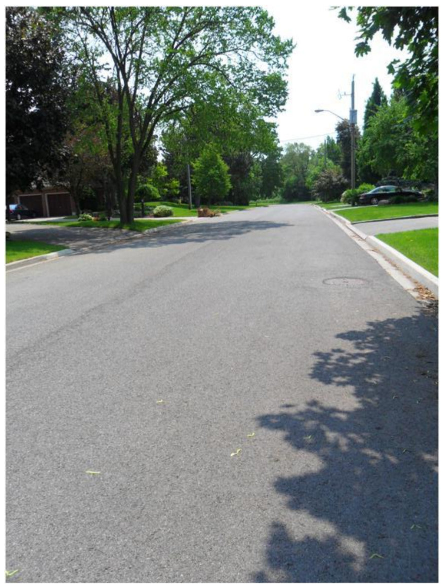



Figure 4-27: Site 2 – Ravelling/Aggregate Loss along the centreline

Figure 4-28: Site 2 - Transverse Cracking

Also, hairline cracks were observed at the start of Vintage Lane (7.1m by 4.0m). The street consisted of a section of frequent ravelling along the centreline with an average area of $163m^2$.

The pavement exhibited similar and consistent surface and structural integrity as previously observed in 2010 and shown in Figure 4-29. Hence, the street pavement was still in an excellent condition to support the designed for traffic load.

Figure 4-29: Site 2 - Comparison of Surface Characteristics

4.3.3.3 ***Site 3: Thornhill Summit Drive***

The street was observed to be in better condition compared to Ida Street exhibiting comfortable (RCR value – 8) pavement surface when related to the Ride Condition Rating (RCR) Scale used by MTO. It exhibited slight intermittent wheel track rutting, and multiple longitudinal cracking, moderate frequent multiple centerline cracking, and very slight intermittent transverse cracking and raveling/aggregate loss as shown in Figure 4-30 to Figure 4-34. Overall the pavement is in excellent performing condition except for the two sections with construction equipment damage (forklift).

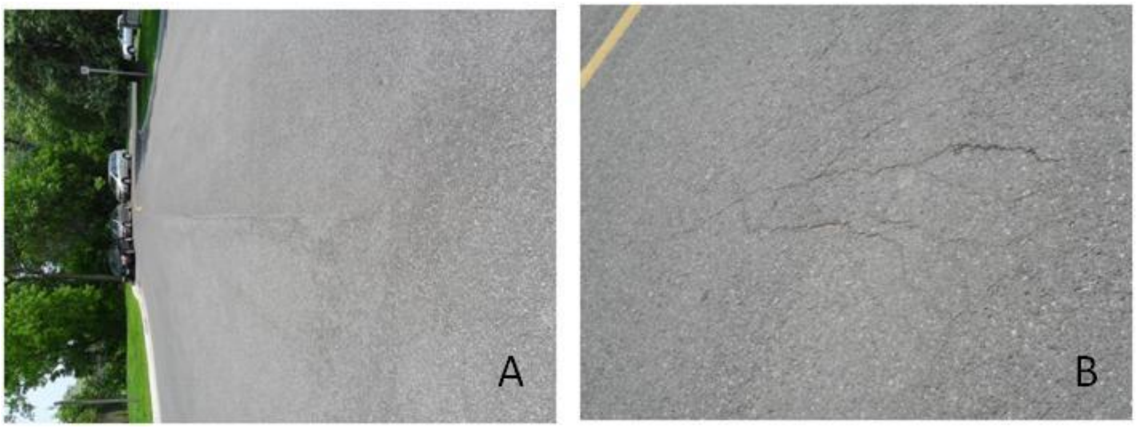


Figure 4-30: Site 3 – Longitudinal (A) and Transverse (B) Cracking

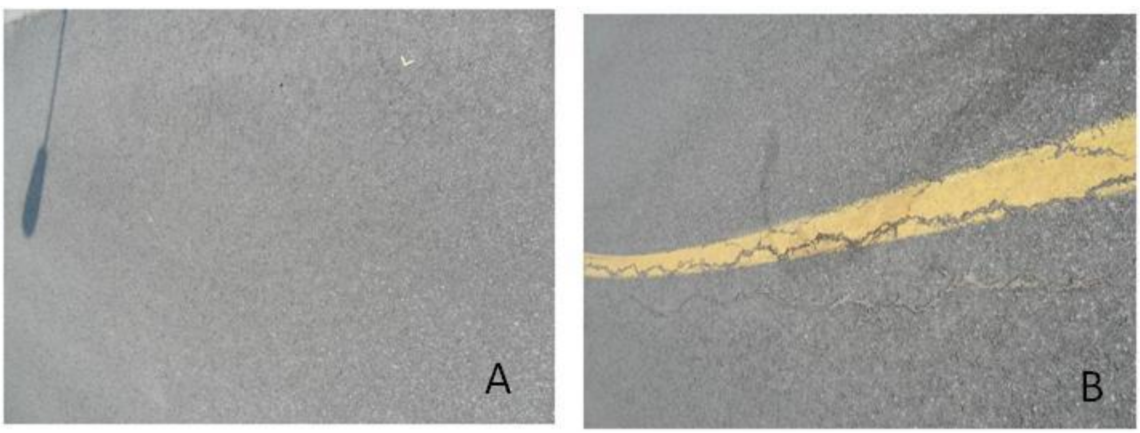


Figure 4-31: Site 3 – Slight Pop-outs (A) and Centreline Cracking (B)

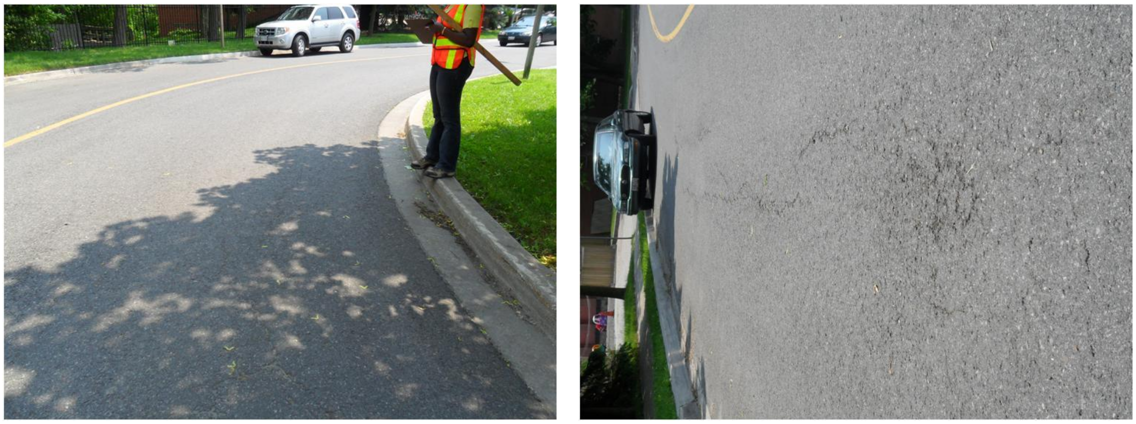


Figure 4-32: Site 3 – Ravelling/Aggregate Loss

Figure 4-33: Site 3 – Slight Rutting

Figure 4-34: Site 3 – Construction Equipment Damage

The pavement exhibited similar and consistent surface and structural integrity as previously observed in 2010 and shown in Figure 4-35 hence the pavement is still in good condition to serve the designed purpose.

Figure 4-35: Site 3 - Comparison of Surface Characteristics

Overall the three residential streets under study exhibit no significant signs of surface distresses and are in good condition with Paul Street and Vintage Lane exhibiting the best performance followed by Thornhill Summit Drive. The storm management drains on the pavements are still in excellent condition without any blockage.

4.4 Summary

This chapter summarizes the results of the laboratory and field evaluation on RAS pavements. The various results from freeze-thaw analysis and field surveys have been presented. Pavements constructed with less than 3% RAS content in the HMA were observed to perform similarly to Conventional HMA pavements.

Chapter 5: Structural Evaluation of RAS Pavement

5.1 Introduction

This chapter discusses the performance prediction modeling performed on all the six mixes researched in this thesis using the MEPDG software. Historically; asphalt pavement thickness was designed based on vehicle type, standardized axle loads, and material properties however, in recent years; pavement design has shifted towards a mechanistic-empirical framework. The framework focuses on engineering principles to design pavement structures that can resist specific distresses such as fatigue cracking and rutting over the analysis performance period. The mechanistic-empirical design incorporates material properties and environmental data and uses mechanical properties to more accurately model pavement structures [NCAT 2010]. The analysis in the study used the dynamic modulus, asphalt binder properties, and material gradation. Validation of the performance prediction models of the design mixes with recycled material against Conventional HMA were also carried out for this chapter.

5.2 Pavement Response Model Results (MEPDG)

Pavement responses are calculated based on the expected traffic loading, which is then used to predict pavement performance through empirical correlations. MEPDG provides reliable model outputs on the pavement performance over the design analysis period. This research focused on a 20 year analysis period to evaluate the pavement performance for all the six mixes. The distresses indicated from the analysis included surface down cracking (longitudinal cracking), bottom up cracking (alligator cracking), surface down and bottom up damages, roughness (IRI), and total pavement and AC deformation (rutting). Minimal damage was noticed in all layers of the pavement structure. Better performance was observed for mixes without the addition of RAP compared to the mixes with RAP percentages at 70% reliability. SP 19 6% RAS had the best overall performance in all the distresses followed by SP 12.5 FC 2 6% RAS as given in Table 5-1.

Transverse cracking was not accounted for in this study as it can normally be addressed through the broken sections of the HMA layer. However, from the field evaluations, it was observed that all the sections had very slight to slight intermittent transverse cracking. For the Town of Markham, three streets have undergone four winters while CPATT Test Track has undergone two winters. It should be noted that these test sections contain Mix 1: HL 3 1.5%RAS and 13.5% RAP (Paul Street and

Vintage Lane, Thornhill Summit Drive, and CPATT Test Track) and SP12.5 FC1 3.5% RAS (Ida Street).

Table 5-1: Reliability Summary of Performance Predictions

Performance Criteria			Control Mix: Conventional HL3			Mix 1: HL3 1.5% RAS 13.5%			Mix 2: SP 19 6% RAS			Mix 3: SP 19 3% RAS	
Distresses Assessed	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable	Distress Predicted	Reliability Predicted	Acceptable	Distress Predicted	Reliability Predicted	Acceptable	Distress Predicted	Reliability Predicted
Terminal IRI (mm/m)	3	70	1.85	98.77	Pass	2.01	96.49	Pass	1.79	99.25	Pass	1.89	98.29
AC Surface Down Cracking (Long. Cracking) (m/km):	10560	70	29832.00	6.59	Fail	35164.80	2.80	Fail	28564.80	7.92	Fail	33105.60	3.96
AC Bottom Up Cracking (Alligator Cracking) (%):	25	70	4.90	92.25	Pass	12.90	80.41	Pass	2.90	94.18	Pass	7.40	89.37
Permanent Deformation (AC Only) (mm):	6	70	5.84	60.60	Fail	8.13	24.25	Fail	4.57	87.95	Pass	6.10	53.00
Permanent Deformation (Total Pavement) (mm):	19	70	14.73	95.17	Pass	18.03	62.16	Fail	12.95	99.47	Pass	15.49	90.03

Performance Criteria			Control Mix: Conventional HL3			Mix 4: SP 12.5 FC1, 3% RAS and			Mix 5: SP 12.5 FC2, 6% RAS			Mix 6: SP 12.5 FC2, 3'	
Distresses Assessed	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable	Distress Predicted	Reliability Predicted	Acceptable	Distress Predicted	Reliability Predicted	Acceptable	Distress Predicted	Reliability Predicted
Terminal IRI (mm/m)	3	70	1.85	98.77	Pass	1.97	97.27	Pass	1.90	98.24	Pass	1.96	97.40
AC Surface Down Cracking (Long. Cracking) (m/km):	10560	70	29832.00	6.59	Fail	33422.40	3.76	Fail	32630.40	4.27	Fail	33105.60	3.96
AC Bottom Up Cracking (Alligator Cracking) (%):	25	70	4.90	92.25	Pass	10.00	85.59	Pass	7.20	89.57	Pass	9.50	86.41
Permanent Deformation (AC Only) (mm):	6	70	5.84	60.60	Fail	7.87	27.58	Fail	6.60	47.50	Fail	7.62	28.04
Permanent Deformation (Total Pavement) (mm):	19	70	14.73	95.17	Pass	17.53	69.87	Fail	15.75	88.04	Pass	17.27	71.18

5.2.1 Performance Prediction

Pavement material tends to deteriorate under the influence of traffic loads and climatic effects resulting in microcracking in the asphalt materials and also pavement deformation due to the stresses accumulated with time. The micro-cracking may result in loss of skid resistance (safety) due to changes in surface texture and aggregate polishing/bleeding, and comfort for road users. Pavement performance models facilitate in capturing the process of deterioration of the pavement in a comprehensive manner considering all the influencing factors such as traffic loading and climate. However, the material deterioration process is quite complex and difficult to predict [FordFoU 2006].

The MEPDG pavement design guide was used to model pavement performance putting into consideration the influence of traffic loads and climatic effects. The service life of the design mixes was analyzed based on longitudinal and alligator cracking, rutting and IRI as given in Table 5-2 and Table 5-3. It was observed that binder layer mix SP19 6% RAS had the best performance predictions while SP12.5 FC2 6% RAS had better performance among the surface layer mixes. SP19 6% RAS performed better than conventional HL 3.

Table 5-2: Pavement Performance Prediction Matrix (Service Life for Mixes)

Mix Description	Pavement Service Life (Years)				
	Longitudinal Cracking	Fatigue Cracking	Total Pavement Rutting	Asphalt Cement Rutting	International Roughness Index (IRI)
	1000m/km	10%	13mm	4mm	2mm/m
Control: Conventional HL 3	3.8	12.8	8.8	9.8	17.5
Mix 1: HL 3 1.5% RAS 13.5% RAP	3.0	6.0	4.0	4.9	14.0
Mix 2: SP19 6% RAS	4.1	19.0	14.0	15.0	19.0
Mix 3: SP19 3% RAS 25% RAP	3.6	9.0	7.0	8.8	16.5
Mix 4: SP12.5 FC1 3% RAS 17% RAP	3.5	7.0	4.8	5.8	14.8
Mix 5: SP12.5 FC2 6% RAS	3.5	9.0	6.5	7.8	16.5
Mix 6: SP12.5 FC2 3% RAS 12% RAP	3.5	7.5	4.8	5.8	14.8

Table 5-3: Ranking of Pavement Design Mix considering Service Life

Mix Description	Pavement Service Life (Years)					
	Longitudinal Cracking	Fatigue Cracking	Total Pavement Rutting	Asphalt Cement Rutting	International Roughness Index (IRI)	Rank (1-Best, 7-Worst)
	1000m/km	10%	13mm	4mm	2mm/m	
Control: Conventional HL 3	2	2	2	2	2	2
Mix 1: HL 3 1.5% RAS 13.5% RAP	7	7	7	7	7	7
Mix 2: SP19 6% RAS	1	1	1	1	1	1
Mix 3: SP19 3% RAS 25% RAP	3	3	3	3	3	3
Mix 4: SP12.5 FC1 3% RAS 17% RAP	4	6	5	5	5	6
Mix 5: SP12.5 FC2 6% RAS	4	3	4	4	3	4
Mix 6: SP12.5 FC2 3% RAS 12% RAP	4	5	5	5	5	5

5.2.2 Performance Prediction for Surface Layers Mixes

The surface layer mixes consisted of three mixes and these were compared against two control mixes; Control Mix: Conventional HL 3 and Mix 1:HL 3 1.5%RAS, 13.5%RAP which has been previously used on pavements under study as illustrated in Figure 5-1 to Figure 5-6. For all the distresses, it was observed that overall mixes with only RAS or no recycled material performed better compared to the ones with both RAS and RAP. The mixes with both RAS and RAP exhibited similar performance response to Mix 1 as shown in proceeding figures and discussion below. Overall for surface layer mixes, SP12.5 FC2 6% RAS exhibited better performance and similarly to conventional HL 3 while SP12.5 FC1 3%RAS, 17% RAP and SP12.5 FC2 3% RAS, 12% RAP had similar if not better performance response to Mix 1.

Longitudinal Cracking

Longitudinal cracking or fatigue cracking is parallel to the pavement centreline and located in the right wheel path. It allows moisture infiltration, roughness, structural failure and possible onset for alligator cracking. The surface layer mixes exhibited slower rates of deterioration or surface-down damage during the analysis period, with the three alternative mixes reaching maximum damage(100%) a year after Mix 1 as shown in Figure 5-1. However, it is advisable to apply some form of treatment to the pavement in order to preserve the facility as well as to offer service to the user as designed for. Preservation pavement treatment can be started as early as after three to five years of construction. The terminal surface-down cracking value is reached between three to four years as illustrated by Figure 5-2.

Conventional HL 3 is observed to perform better than mixes with RAS and/or RAP reaching its terminal serviceability value a year later than the alternative mixes.

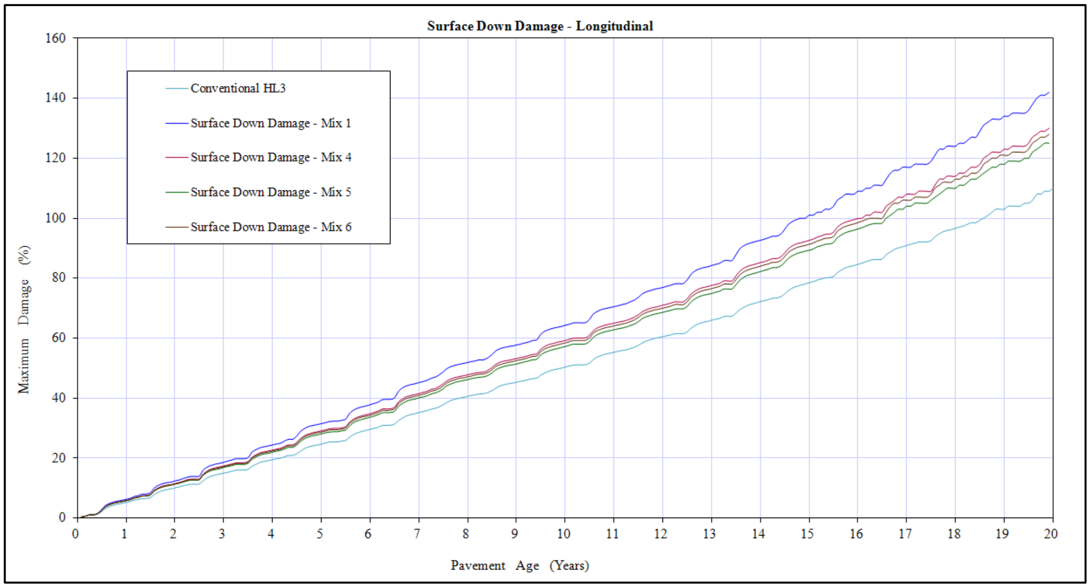


Figure 5-1: Surface-Down (Longitudinal) Damage Surface Layer Mix Comparisons

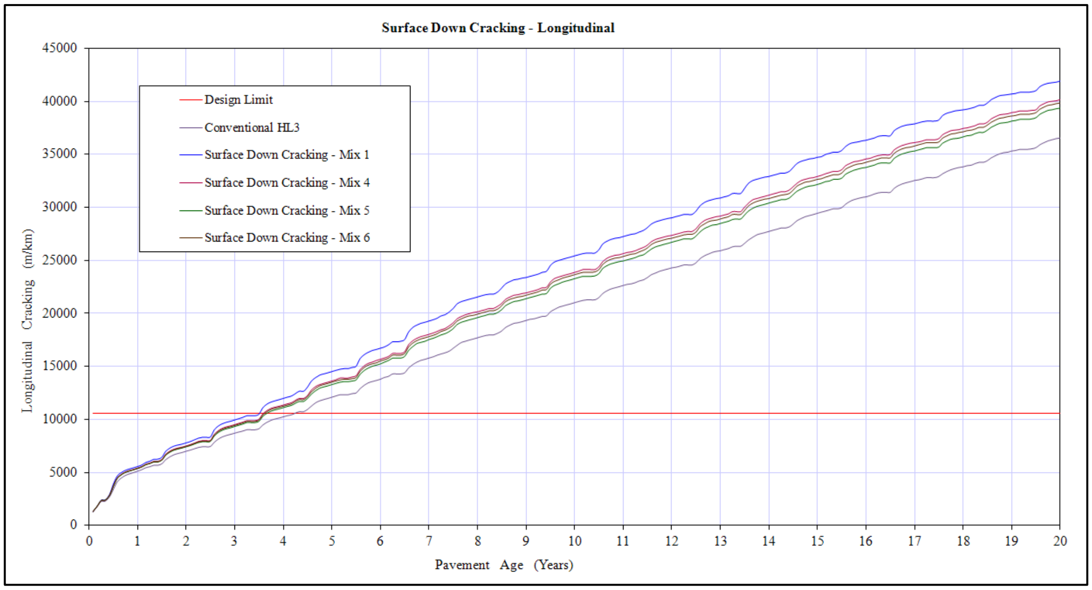


Figure 5-2: Surface-Down Cracking (Longitudinal) Surface Layer Mix Comparisons

Alligator Cracking

Alligator cracking illustrates interconnected cracks caused by fatigue failure of the HMA surface under repeated traffic loading. For the study, the model analyzed bottom-up cracking characteristics in the surface layer HMA mixtures, the bottom of the HMA layer is expected to incur higher tensile stresses, hence cracks tend to propagate from that point. Alligator cracking is usually due to structural failure and will result in moisture infiltration, roughness, and eventually develop into potholes.

However, for all the surface layer mixes, no signs of alligator cracking were observed as illustrated in Figure 5-3 and Figure 5-4 over the design period. Mix 5: SP12.5 6% had the best resistance to bottom-up cracking while Mix 1 had the least resistance to cracking when compared to conventional HL 3. Mix 4: SP12.5 FC1 3% RAS, 17% RAP and Mix 6: SP12.5 FC2 3% RAS, 12% RAP were observed to exhibit similar response and closer to Mix 1 responses. There is hardly any cracking observed within the first three years while a very slow rate of deterioration is observed between three to six years and then the rate of deterioration completely stays constant for the preceding years as illustrated in Figure 5-4.

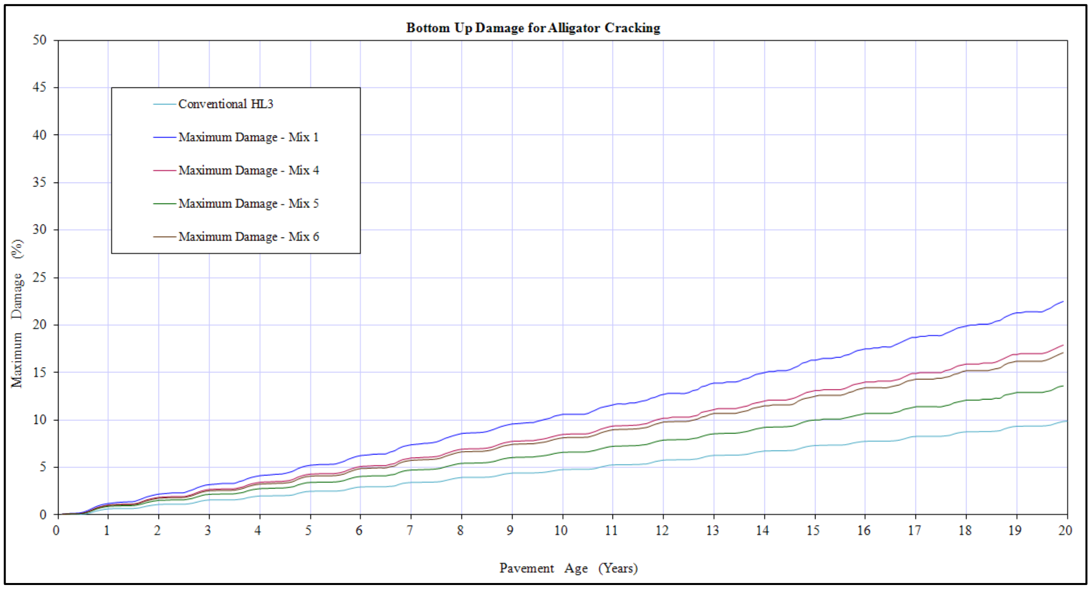


Figure 5-3: Bottom-Up (Alligator) Damage Surface Layer Mix Comparison

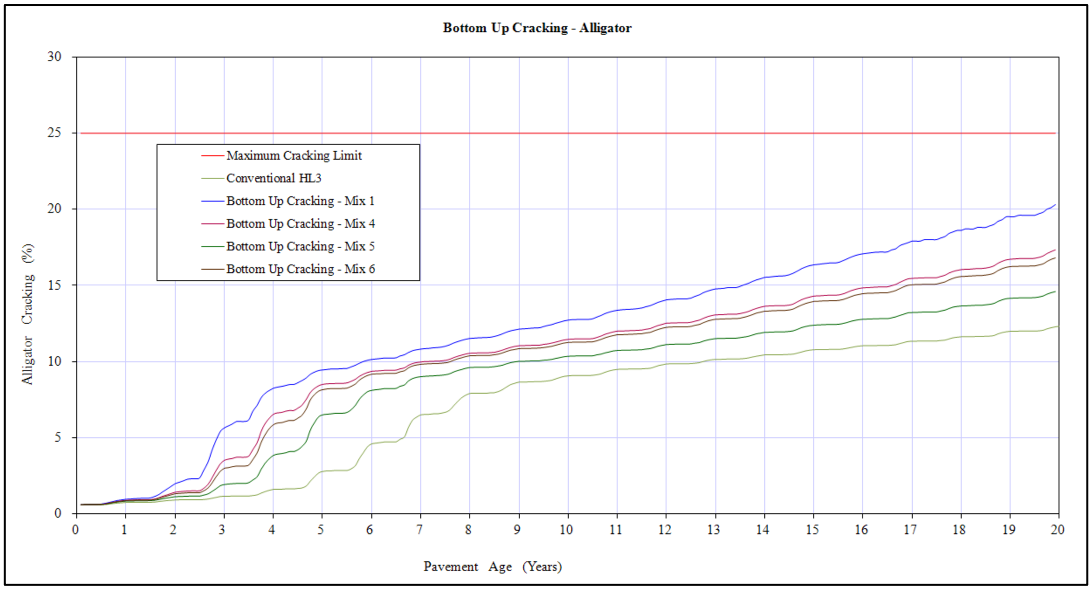


Figure 5-4: Bottom-Up Cracking (Alligator) Surface Layer Mix Comparison

Permanent Deformation (Rutting)

Ruts are surface depressions in the pavement wheel paths usually caused by consolidation or lateral movement due to traffic loading. Rutting can also be due to inadequate pavement structure (in case of subgrade rutting) or improper mix design (in case of HMA layer). Rutting prediction over the analysis period is illustrated by Figure 5-5. Prevention measures when taken can preserve the pavement to a better condition.

- Total Pavement Rutting; Mix 5 exhibited no signs of rutting throughout its design life with a 17mm rut depth at year 20 closer to conventional HL 3. Mixes 4 and 6 reached their terminal rut depth (19.05mm) after the 20th year and Mix 1 had its terminal rut depth value at 17years.
- Asphalt Cement (AC) Rutting; Mix 5 exhibited better resistance to rutting reaching its terminal serviceability value at 19years while mixes 1, 4, and 6 exhibited terminal service value between 12years and 13years. Mix 1 demonstrated the least resistance to rutting in both the AC layer and total pavement.

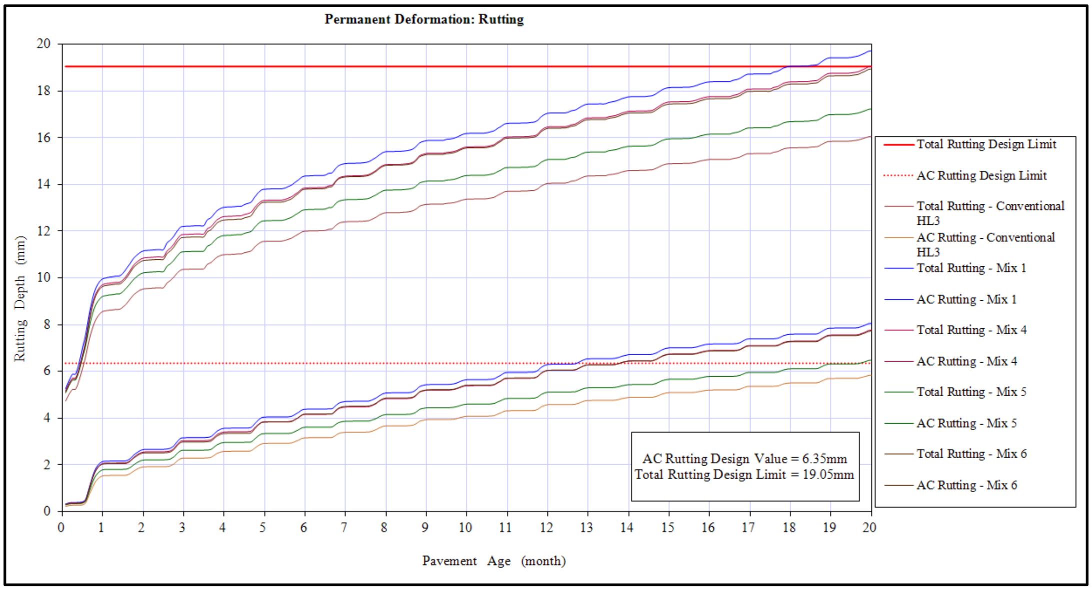


Figure 5-5: Permanent Deformation (Rutting) Surface Layer Mix Comparison

Pavement Smoothness

Roughness is the distortion of the pavement surface that contributes to an undesirable or uncomfortable ride. Roughness is an important indicator of the pavement riding comfort and safety [TAC 1997]. It can be quantified in terms of the International Roughness Index (IRI) which relates to the Ride Comfort Index (RCI). The IRI gives the comfort level and safety of a pavement for the road users. Roughness can be caused by traffic loading, environmental effects, construction materials or built-in construction irregularities. It tends to increase with exposure to traffic loading manifested by corrugations that cause an increase in dynamic wheel force increasing severity, and environment through poor drainage, swelling soils, freeze-thaw cycles, and non-uniform consolidation of the subgrade [Shahin 2005].

All the surface layer mixes had an IRI index way below the terminal serviceability value in millimetre per meter (mm/m) as illustrated by Figure 5-6. All the design mixes were observed to exhibit no significant signs of roughness for almost 12years with Mix 1 reaching 2mm/m value at 14years.

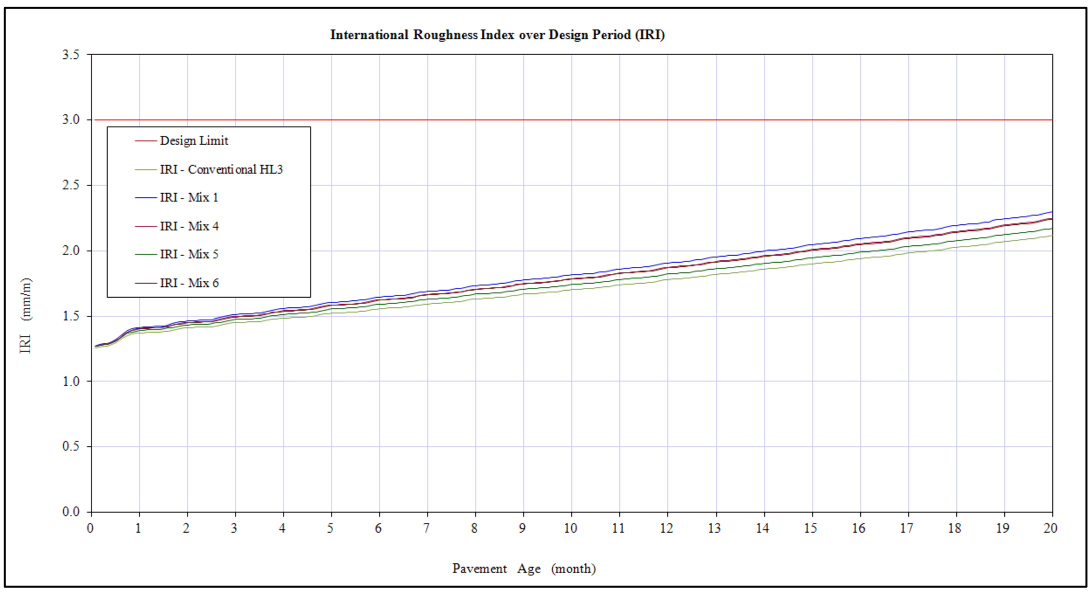


Figure 5-6: IRI over Design Period for Surface Layer Mix

5.2.3 Performance Prediction for Binder Layers Mixes

The binder layer mixes which contain a 19mm stone and are denoted as Mix 2 and Mix 3 are both Superpave SP19E. The performance of the two mixes was compared to both the control mix, Conventional HL 3 and Mix 1 to illustrate relative performance. Mix 2: SP19 6% RAS illustrated better performance characteristics compared to both the control mix and Mix 1 given in Table 5-3. This could be related to the fact that the RAS is better able to mix with the larger aggregate.

Longitudinal Cracking

Mix 2 was observed to exhibit less pavement damage throughout the analysis period and barely reached 100% damage at its 20th year whereas Mix 1 and Mix 3 reached 100% damage at 15years and 14years respectively as shown in Figure 5-7. At 50% pavement damage due to longitudinal cracking was observed between 7years and 12years for the mixes.

As illustrated in Figure 5-8, Mix 1 and Mix 3 reached their terminal serviceability value at 3.5years while Mix 2 reached its terminal serviceability value after 4years similar to conventional HL 3. A preservation treatment such as routine and preservative maintenance should be planned as early as 3years to extend pavement service life. This pavement treatment would enable for crack repairs such as rout and seal treatment resulting in extension of service life.

Alligator Cracking

All the mixes illustrated little or no significant signs of alligator cracking and damage. The total damage percentage over the analysis period was below the terminal serviceability value of 25% as shown in Figure 5-10, indicating that RAS mixes had a higher potential to resist fatigue cracking similarly to conventional HMA mixes.

Mix 2 had minimal cracking beyond 8 years and this occurred at a slower rate as shown in Figure 5-9. Mix 2 was observed to perform better than conventional HL 3 over a 20year design life.

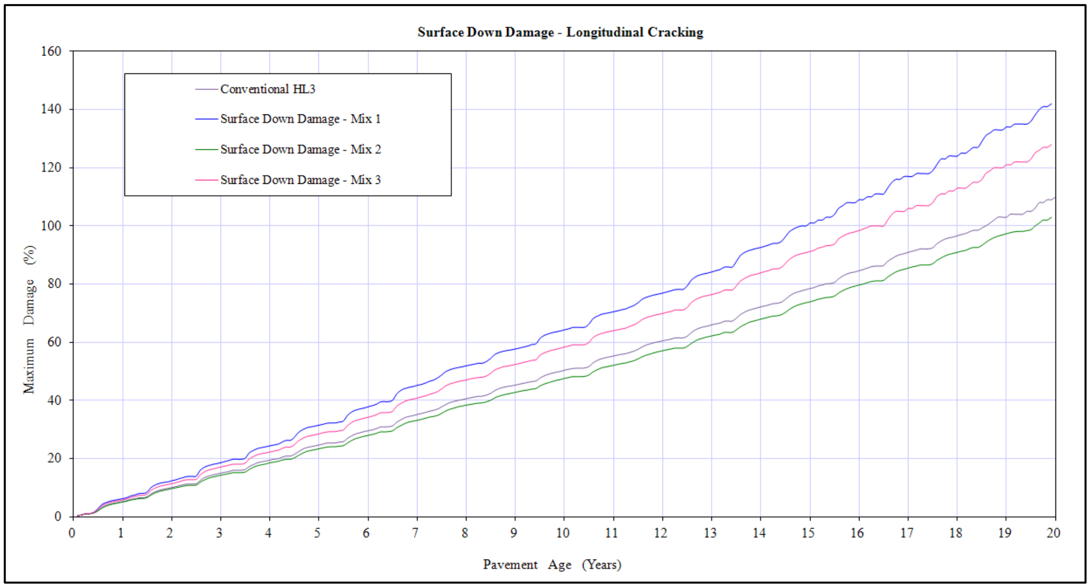


Figure 5-7: Surface-Down (Longitudinal) Damage Binder Layer Mix Comparisons

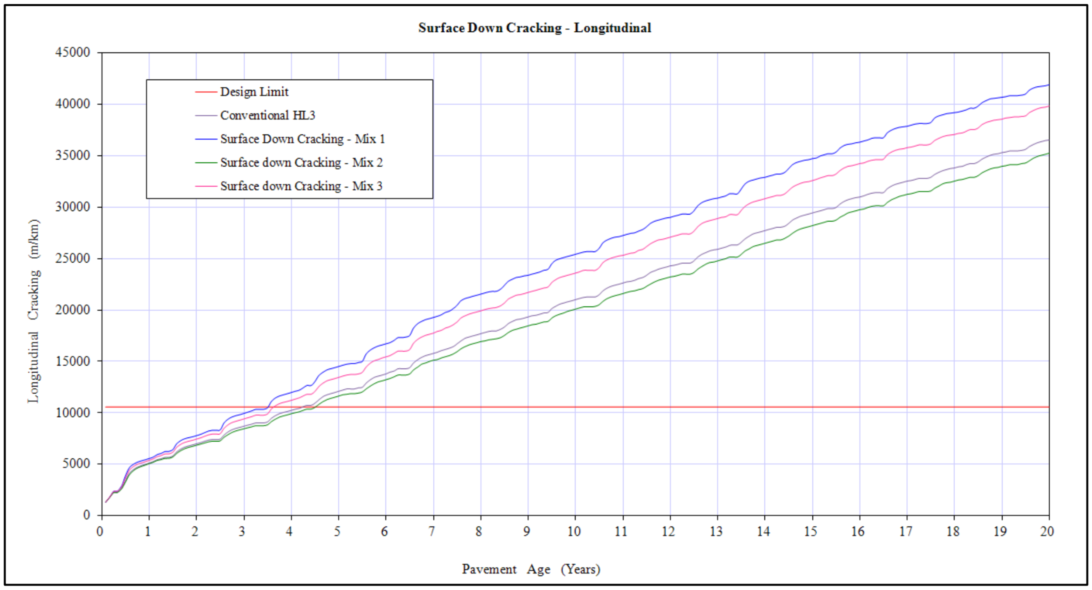


Figure 5-8: Surface-Down Cracking (Longitudinal) Binder Layer Mix Comparisons

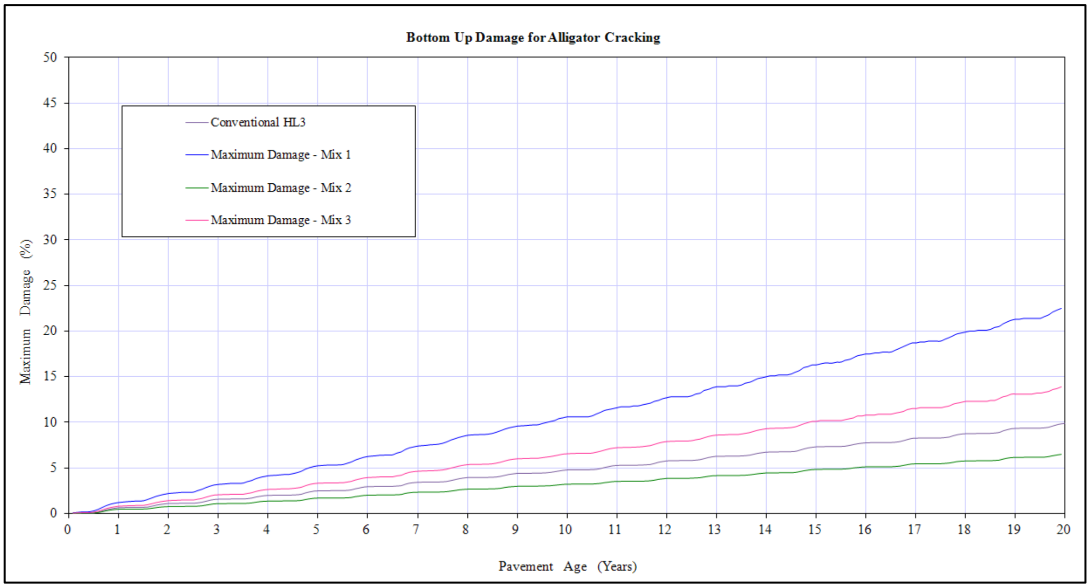


Figure 5-9: Bottom-Up (Alligator) Damage Binder Layer Mix Comparison

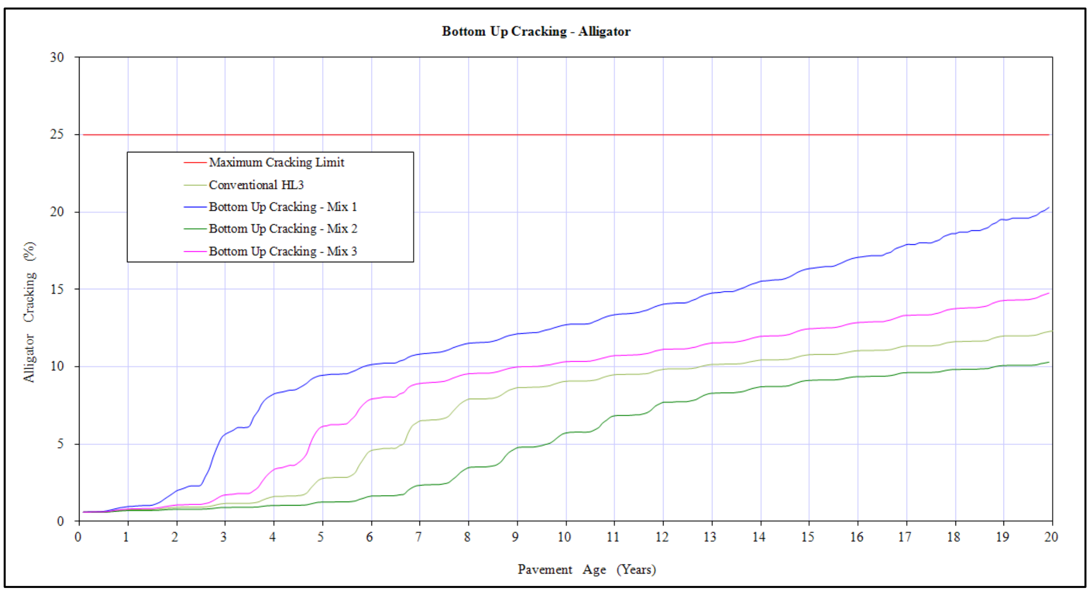


Figure 5-10: Bottom-Up Cracking (Alligator) Binder Layer Mix Comparison

Permanent Deformation (Rutting)

Mix 2 and Mix 3 did not have high density of rutting in either the total pavement structure or the asphalt cement (AC) layer as indicated in Figure 5-11. Both mixes performed much better than Mix 1, Mix 3 was predicted to perform similarly to the control, Conventional HL 3 in rutting resistance whereas Mix 2 was predicted to perform better than conventional HL 3 and still in good condition by the end of the analysis period. However, prevention measures should be taken to extend the pavement service life by applying preservation treatments between six and nine years.

Pavement Smoothness

All mixes illustrated good IRI characteristics with a smoothness of less than 2.5mm/m over the design/analysis period. Both mixes performed better than Mix 1 and performed similarly to control mix, Conventional HL 3 as shown in Figure 5-12, though Mix 2 still showed exceptionally better performance prediction.

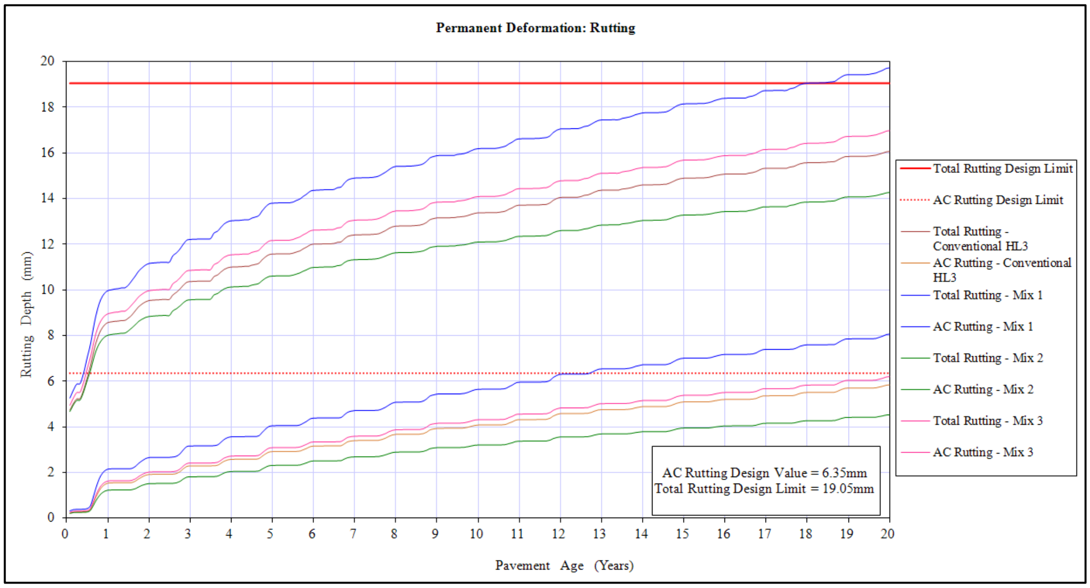


Figure 5-11: Permanent Deformation (Rutting) Binder Layer Mix Comparison

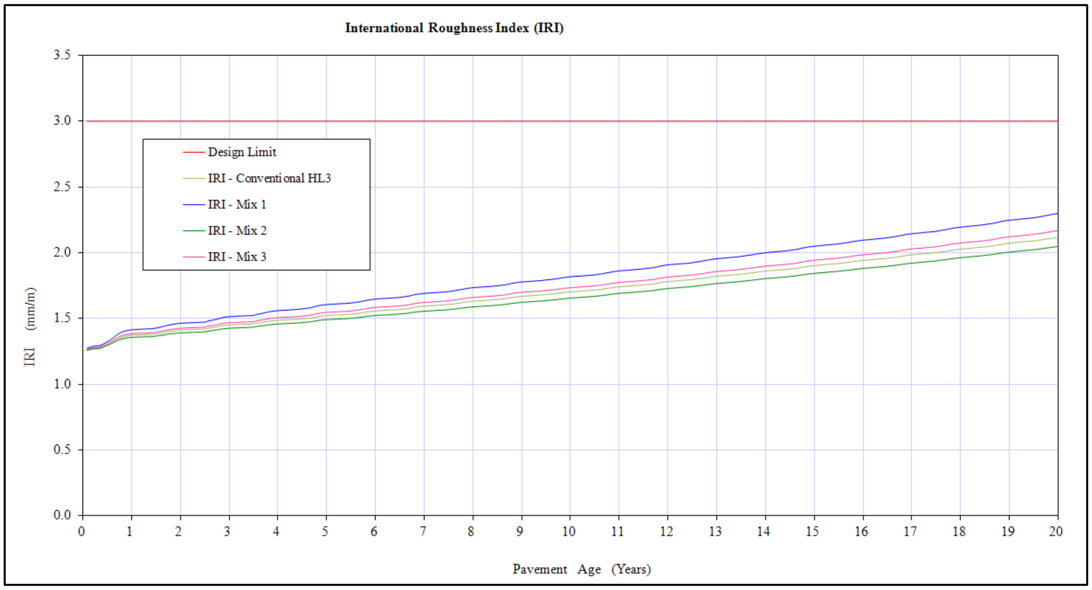


Figure 5-12: IRI over Design Period for Binder Layer Mix

Overall Mix 2: SP19 6% RAS illustrated greater potential at resisting longitudinal cracking, alligator cracking, pavement damage (rutting), and pavement roughness when compared with conventional HL 3, and other RAS/RASP mixes.

5.3 Validation of the Performance Prediction Models

A statistical analysis was performed to compare the control mix Conventional HMA mix to other mixes in terms of individual distresses. All statistical tests were performed at a 95% confidence level and the strength of favoring the alternative mix instead of the control mix, Conventional HL 3 lies in rejecting the null hypothesis. All the distress prediction models were analyzed at a 70% reliability level, which is consistent with standard Ontario practice for arterial and collector roads and also the same reliability used in the MEPDG analysis.

The t-test as shown in the Appendix E indicated that all the alternative mixes (Mixes 2, 3, 4, 5, and 6) behaved differently from both control the mix and Mix 1, and generally all five mixes attained distresses or failure much later than Mix 1. The P-Value attained in all the statistical analysis was less than 0.001 indicating strong evidence in favor of the alternative mixes than Mix 1: HL 3 1.5% RAS, 13.5% RAP. Hence increasing the quantity of RAS in HMA mixes tends to improve pavement performance.

5.3.1 Prediction Model for Surface-Down Damage

From the F-test carried out on all the mixes in comparison to Mix 1, indicated that mixes containing RAP had no statistically significant variation from Mix 1 as demonstrated by $F_{calculated}$ value being less than the $F_{critical}$ value. Hence they exhibited similar surface down damage in the pavement over the analysis period to Mix 1 as given in Table 5-4. Mixes formulated with only RAS as an addition to HMA, exhibited statistically significant variation from those with both RAS and RAP in the surface-down damage incurred by the pavement. Mix SP 19 6% RAS exhibited the highest variation from Mix1 as illustrated by the P-Value, which was less than 0.001.

However, the rate of deterioration for the alternative mixes was consistent with that of Mix 1 as in Figure 5-13. It was observed that there was a slight difference in the rate of deterioration of the average alternative mixes (R^2 = 99%) from Mix 1 (R^2 = 99.6%). Mix 1 was observed to exhibit earlier surface-down damage than the five alternative design mixes.

Table 5-4: Surface-Down Damage Statistical Comparison to Mix 1

F-Test Two-Sample for Variances

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	66.67	48.95
Variance	1649.65	866.35
Observations	240	240
df	239	239
F	**1.90**	
P(F<=f) one-tail	4.09E-07	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	66.67	60.37
Variance	1649.65	1344.81
Observations	240	240
df	239	239
F	**1.23**	
P(F<=f) one-tail	0.06	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	66.67	61.33
Variance	1649.65	1386.91
Observations	240	240
df	239	239
F	**1.19**	
P(F<=f) one-tail	0.09	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	66.67	59.18
Variance	1649.65	1285.75
Observations	240	240
df	239	239
F	**1.28**	
P(F<=f) one-tail	0.03	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	66.67	60.46
Variance	1649.65	1345.54
Observations	240	240
df	239	239
F	**1.23**	
P(F<=f) one-tail	0.06	
F Critical one-tail	**1.24**	

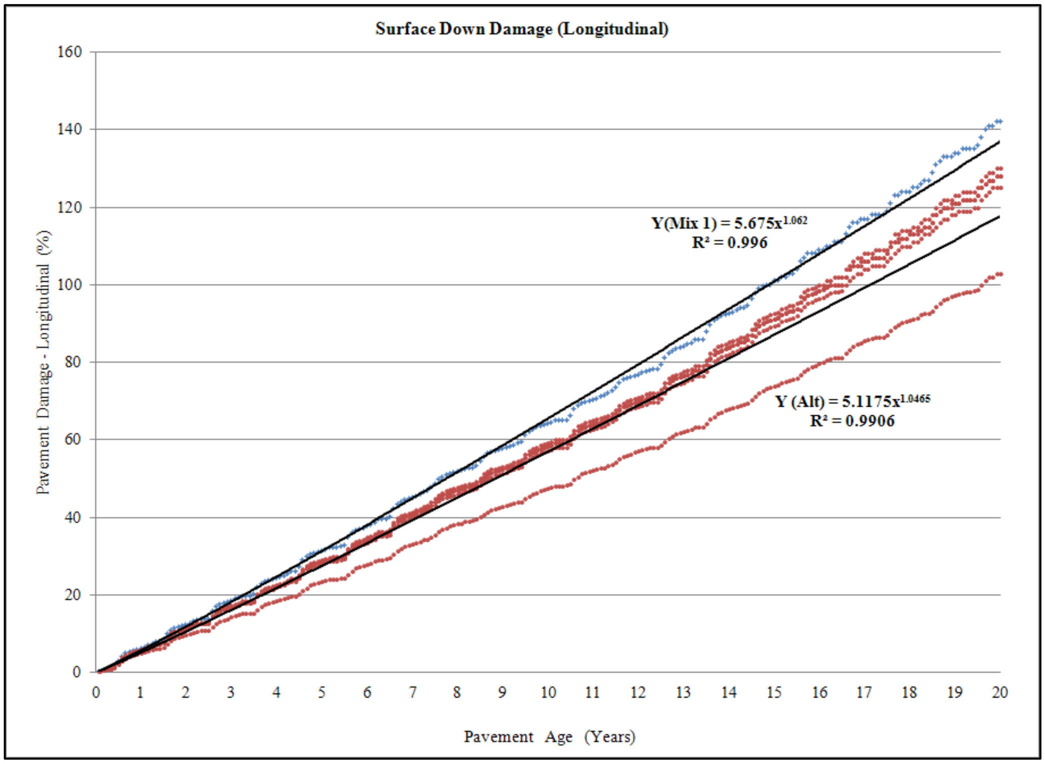


Figure 5-13: Surface-Down Damage Comparison to Mix 1

When compared to the control, Conventional HL 3, all mixes were observed to exhibit no statistically significant differences from the conventional HMA hence they would be expected to have similar amounts of surface down cracking in the field. This is given in Table 5-5 giving the results of the statistical analysis. Hence HMA mixes with RAS and/or RAP will exhibit the same trend of surface-down damage over the analysis period, supported by an average R-value (R^2 = 99.7%) shown in Figure 5-14.

Table 5-5: Surface-Down Damage Statistical Comparison to the Control Mix

F-Test Two-Sample for Variances

	Conventional HL3	*Mix1: HL3 1.5% RAS 13.5% RAP*
Mean	51.95	66.67
Variance	982.34	1649.65
Observations	240	240
df	239	239
F	**0.60**	
P(F<=f) one-tail	0.00003	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	51.95	61.33
Variance	982.34	1386.91
Observations	240	240
df	239	239
F	**0.71**	
P(F<=f) one-tail	0.00395	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 5: SP12.5 FC2 6% RAS*
Mean	51.95	59.18
Variance	982.34	1285.75
Observations	240	240
df	239	239
F	**0.76**	
P(F<=f) one-tail	0.019	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	51.95	60.46
Variance	982.34	1345.54
Observations	240	240
df	239	239
F	**0.73**	
P(F<=f) one-tail	0.008	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 2: SP19 6% RAS*
Mean	51.95	48.95
Variance	982.34	866.35
Observations	240	240
df	239	239
F	**1.13**	
P(F<=f) one-tail	0.166	
F Critical one-tail	**1.24**	

	Conventional HL3	*Mix 3: SP19 3% RAS 25% RAP*
Mean	51.95	60.37
Variance	982.34	1344.81
Observations	240	240
df	239	239
F	**0.73**	
P(F<=f) one-tail	0.008	
F Critical one-tail	**0.81**	

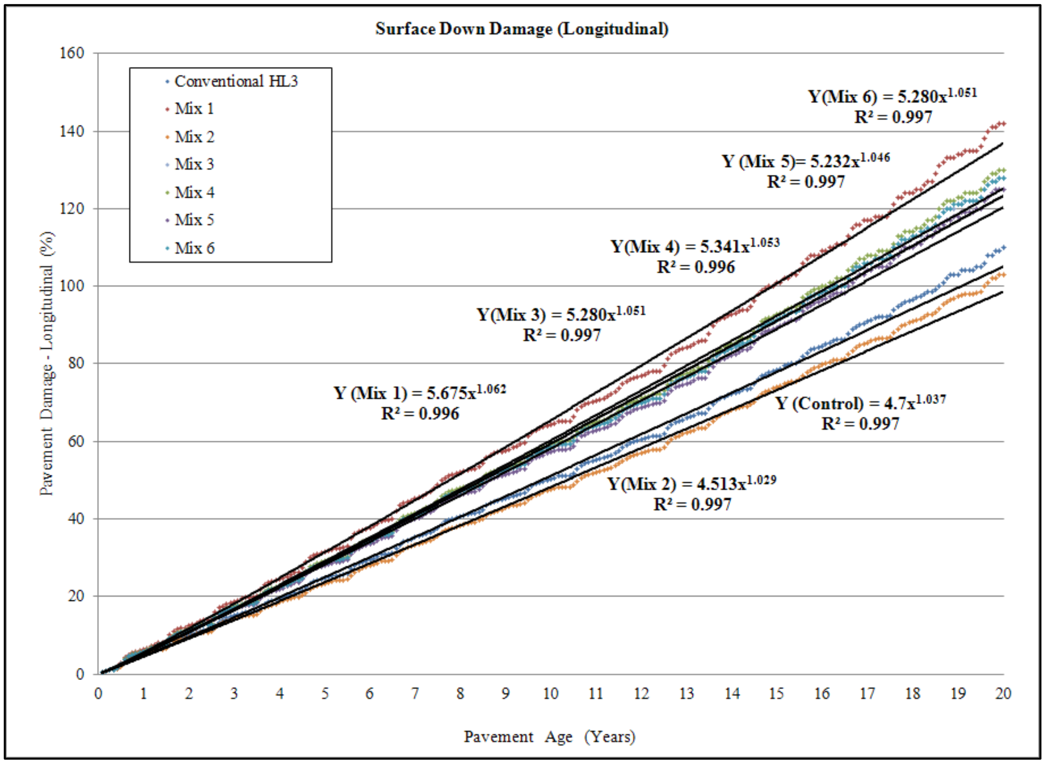


Figure 5-14: Surface-Down Damage Prediction Comparison to the Control Mix

5.3.2 Prediction Model for Surface-Down Cracking (Longitudinal)

The F-test indicated that there was no statistically significant variation in surface down cracking (longitudinal) of the alternative mixes from Mix 1 with RAS except for Mix 2 (SP19 6% RAS) as illustrated by the P-Value being greater than 0.1 as shown in Table 5-6. Hence there was no evidence of variation in longitudinal cracking when compared to Mix 1indicating consistency in rate of cracking for the four mixes with the control mix. However, Mix 2 had a significant variation in longitudinal cracking from HL 3 with RAS. The P-Value (less than 0.001) for Mix 2 indicated very strong evidence of variation, exhibiting better resistance to longitudinal cracking as the pavement ages.

The average rate of cracking (m/km) of the alternative mixes (R^2 = 97.9%) was very similar to Mix 1 (R^2 = 98.9%) as illustrated in Figure 5-15.

Table 5-6: Longitudinal Cracking Statistical Comparison to Mix 1

F-Test Two-Sample for Variances

	Mix1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	24400	19799
Variance	132494380	89069710
Observations	240	240
df	239	239
F	**1.49**	
P(F<=f) one-tail	0.001	
F Critical one-tail	**1.24**	

	Mix1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	24400	22865
Variance	132494380	118267322
Observations	240	240
df	239	239
F	**1.12**	
P(F<=f) one-tail	0.19	
F Critical one-tail	**1.24**	

	Mix1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	24400	23108
Variance	132494380	120306561
Observations	240	240
df	239	239
F	**1.10**	
P(F<=f) one-tail	0.23	
F Critical one-tail	**1.24**	

	Mix1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	24400	22569
Variance	132494380	115036776
Observations	240	240
df	239	239
F	**1.15**	
P(F<=f) one-tail	0.14	
F Critical one-tail	**1.24**	

	Mix1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	24400	22894
Variance	132494380	118289620
Observations	240	240
df	239	239
F	**1.12**	
P(F<=f) one-tail	0.19	
F Critical one-tail	**1.24**	

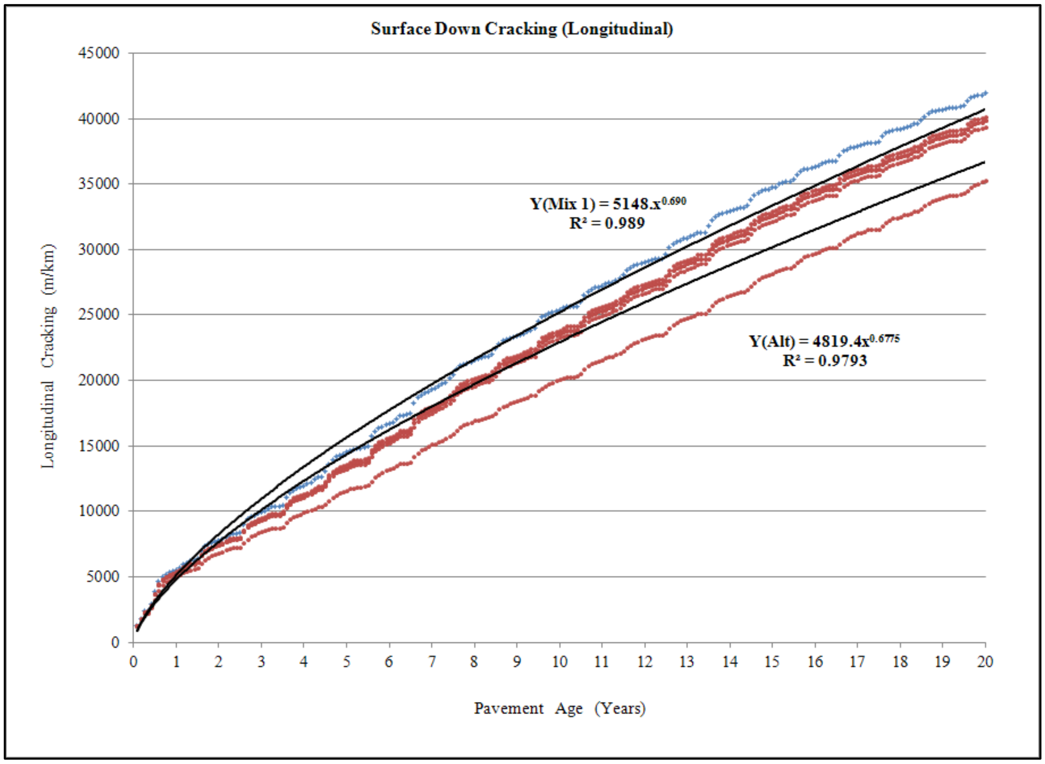


Figure 5-15: Longitudinal Cracking Prediction comparison to Mix 1

A statistical analysis performed on all mixes with RAS and/or RAP indicated that all mixes exhibited cracking before the control, Conventional HL 3 except for Mix 2, which is observed to perform slightly better than the control mix as given in Table 5-7.

The surface layer performed in the way conventional HMA would be expected to perform over a period of time as demonstrated by Figure 5-16, which is supported by a strong R-value (R^2 = 98%). It was observed that Mix 2 had the slowest rate of deterioration in terms of cracking while HL 3 with RAS exhibited the highest rate of cracking relative to conventional HL 3.

Table 5-7: Longitudinal Cracking Statistical Comparison to the Control Mix

F-Test Two-Sample for Variances

	Conventional HL3	*Mix 1: HL3 1.5% RAS 13.5% RAP*
Mean	20642	24400
Variance	97179603	132494380
Observations	240	240
df	239	239
F	**0.73**	
P(F<=f) one-tail	0.008	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	20642	23108
Variance	97179603	120306561
Observations	240	240
df	239	239
F	**0.81**	
P(F<=f) one-tail	0.050	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 5: SP12.5 FC2 6% RAS*
Mean	20642	22569
Variance	97179603	115036776
Observations	240	240
df	239	239
F	**0.84**	
P(F<=f) one-tail	0.096	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	20642	22894
Variance	97179603	118289620
Observations	240	240
df	239	239
F	**0.82**	
P(F<=f) one-tail	0.065	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 2: SP19 6% RAS*
Mean	20642	19799
Variance	97179603	89069710
Observations	240	240
df	239	239
F	**1.09**	
P(F<=f) one-tail	0.251	
F Critical one-tail	**1.24**	

	Conventional HL3	*Mix 3: SP19 3% RAS 25% RAP*
Mean	20642	22865
Variance	97179603	118267322
Observations	240	240
df	239	239
F	**0.82**	
P(F<=f) one-tail	0.065	
F Critical one-tail	**0.81**	

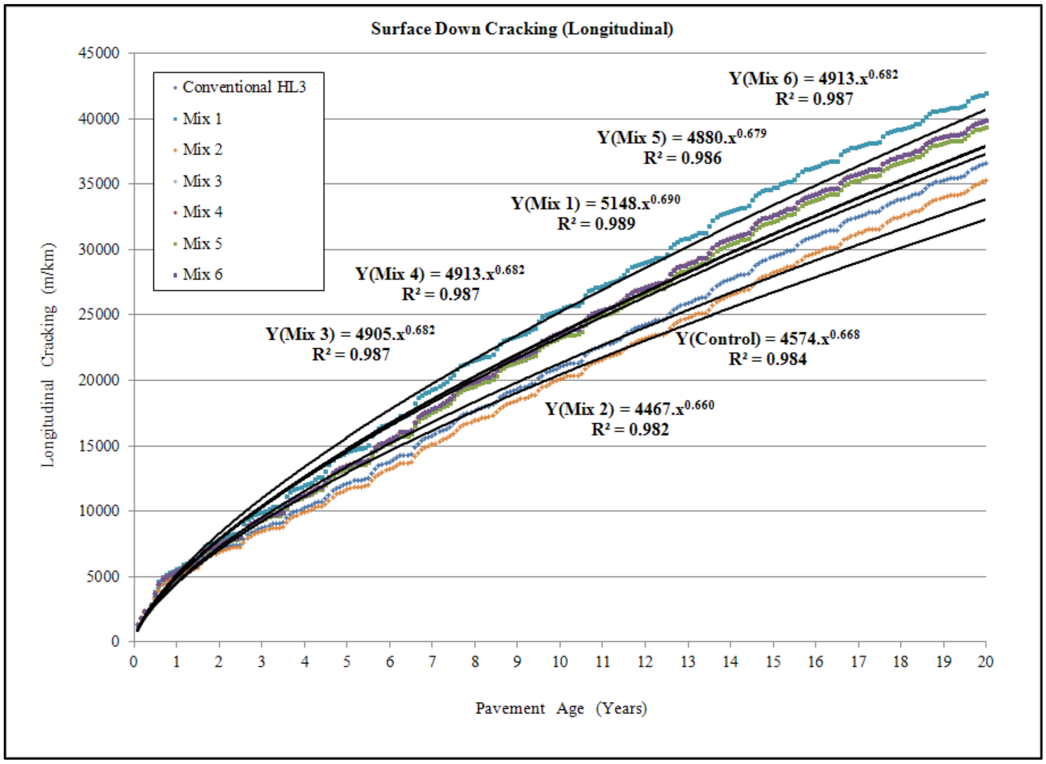


Figure 5-16: Longitudinal Cracking Prediction Comparison to the Control Mix

5.3.3 Prediction Models for Bottom-Up Damage and Bottom-Up Cracking (Alligator)

The F-test performed on mixes for bottom-up damage prediction model indicated that there was very strong evidence of variation in the bottom up damage/cracking of the five alternative mixes from Mix 1. The P-Value (less than 0.001) strongly favors the use of the five alternative mixes compared to Mix 1 in exhibiting better resistance ability to alligator damage in a pavement over the analysis period as given in Table 5-8. Table 5-9 gives the significant variation in alligator cracking of the alternative mixes from Mix 1.

Table 5-8: Bottom-Up Damage Statistical Comparison to Mix 1

F-Test Two-Sample for Variances

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	10.73	3.21
Variance	41.25	3.30
Observations	240	240
df	239	239
F	**12.50**	
P(F<=f) one-tail	2.31E-69	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	10.73	6.64
Variance	41.25	15.44
Observations	240	240
df	239	239
F	**2.67**	
P(F<=f) one-tail	4.96E-14	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	10.73	8.59
Variance	41.25	25.81
Observations	240	240
df	239	239
F	**1.60**	
P(F<=f) one-tail	1.57E-04	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	10.73	6.63
Variance	41.25	14.61
Observations	240	240
df	239	239
F	**2.82**	
P(F<=f) one-tail	2.14E-15	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	10.73	8.21
Variance	41.25	23.60
Observations	240	240
df	239	239
F	**1.75**	
P(F<=f) one-tail	9.10E-06	
F Critical one-tail	**1.24**	

Table 5-9: Bottom-Up Cracking (Alligator) Statistical Comparison to Mix 1

F-Test Two-Sample for Variances

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	11.97	5.30
Variance	30.11	13.36
Observations	240	240
df	239	239
F	**2.25**	
P(F<=f) one-tail	3.00E-10	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	11.97	8.88
Variance	30.11	20.23
Observations	240	240
df	239	239
F	**1.49**	
P(F<=f) one-tail	1.11E-03	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	11.97	10.42
Variance	30.11	23.86
Observations	240	240
df	239	239
F	**1.26**	
P(F<=f) one-tail	0.04	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	11.97	8.92
Variance	30.11	19.39
Observations	240	240
df	239	239
F	**1.55**	
P(F<=f) one-tail	3.55E-04	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	11.97	10.12
Variance	30.11	23.21
Observations	240	240
df	239	239
F	**1.30**	
P(F<=f) one-tail	2.24E-02	
F Critical one-tail	**1.24**	

All the mixes exhibit no signs of bottom up damage over the analysis period as the rate of bottom up damage of the pavement of the alternative mixes (R^2 = 88.2%, R^2 = 80.1%) is slower than Mix 1 (R^2 = 99.3%, R^2 = 92%) as shown in Figure 5-17 and Figure 5-18.

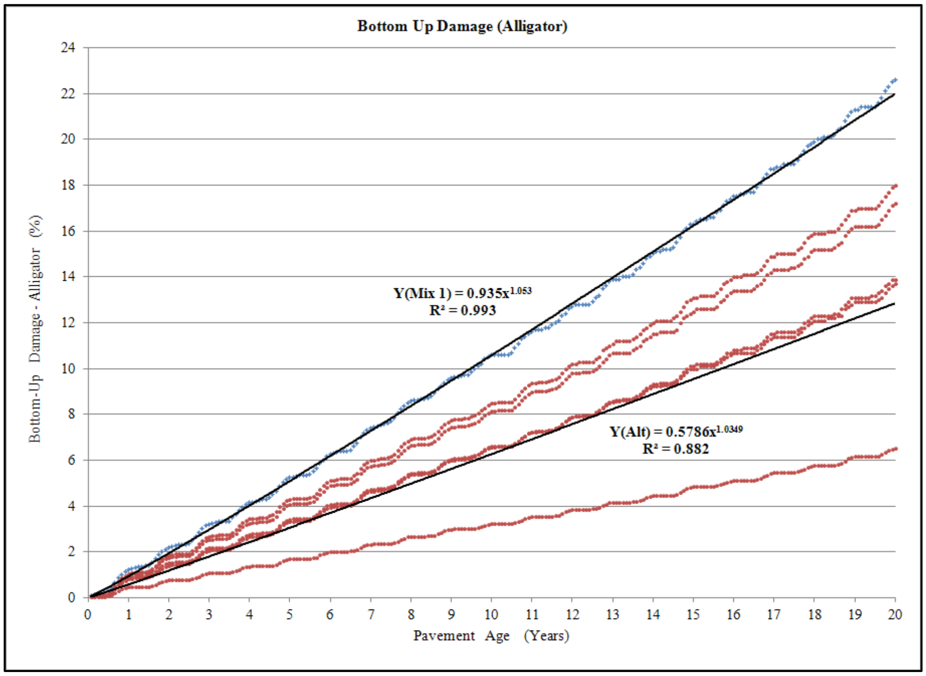


Figure 5-17: Bottom-Up Damage Prediction Comparison to Mix 1

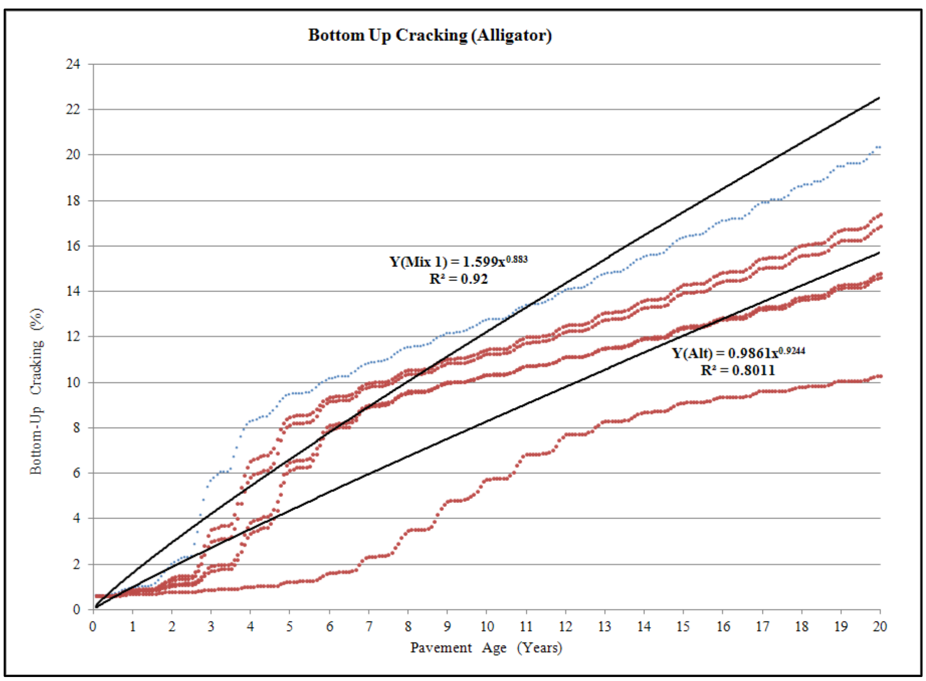


Figure 5-18: Bottom-Up Cracking Prediction Comparison to Mix 1

When compared to Conventional HL 3, all the mixes expect Mix 2 exhibited bottom-up damage or cracking earlier than the control mix as given in Table 5-10. Table 5-11 illustrates that the rate of deterioration is consistent with the control mix, Conventional HL 3 or higher for mixes 1, 4 and 6 while for Mixes 2, 3, and 5 there was significant variation in alligator cracking.

Table 5-10: Bottom-Up Damage Statistical Comparison to the Control Mix

F-Test Two-Sample for Variances: Bottom-Up Damage

	Conventional HL3	*Mix 1: HL3 1.5% RAS 13.5% RAP*
Mean	4.80	10.73
Variance	7.74	41.25
Observations	240	240
df	239	239
F	**0.19**	
P(F<=f) one-tail	0	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	4.80	8.59
Variance	7.74	25.81
Observations	240	240
df	239	239
F	**0.30**	
P(F<=f) one-tail	0	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 5: SP12.5 FC2 6% RAS*
Mean	4.80	6.63
Variance	7.74	14.61
Observations	240	240
df	239	239
F	**0.53**	
P(F<=f) one-tail	5.79E-07	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	4.80	8.21
Variance	7.74	23.60
Observations	240	240
df	239	239
F	**0.33**	
P(F<=f) one-tail	0.00E+00	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 2: SP19 6% RAS*
Mean	4.80	3.21
Variance	7.74	3.30
Observations	240	240
df	239	239
F	**2.35**	
P(F<=f) one-tail	4.37E-11	
F Critical one-tail	**1.24**	

	Conventional HL3	*Mix 3: SP19 3% RAS 25% RAP*
Mean	4.80	6.64
Variance	7.74	15.44
Observations	240	240
df	239	239
F	**0.50**	
P(F<=f) one-tail	6.44E-08	
F Critical one-tail	**0.81**	

Table 5-11: Bottom-Up Cracking Statistical Comparison to the Control Mix

F-Test Two-Sample for Variances: Bottom-Up Cracking

	Conventional HL3	*Mix 1: HL3 1.5% RAS 13.5% RAP*
Mean	7.25	11.97
Variance	16.98	30.11
Observations	240	240
df	239	239
F	**0.56**	
P(F<=f) one-tail	0	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	7.25	10.42
Variance	16.98	23.86
Observations	240	240
df	239	239
F	**0.71**	
P(F<=f) one-tail	0	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 5: SP12.5 FC2 6% RAS*
Mean	7.25	8.92
Variance	16.98	19.39
Observations	240	240
df	239	239
F	**0.88**	
P(F<=f) one-tail	1.53E-01	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	7.25	10.12
Variance	16.98	23.21
Observations	240	240
df	239	239
F	**0.73**	
P(F<=f) one-tail	7.98E-03	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 2: SP19 6% RAS*
Mean	7.25	5.30
Variance	16.98	13.36
Observations	240	240
df	239	239
F	**1.27**	
P(F<=f) one-tail	3.23E-02	
F Critical one-tail	**1.24**	

	Conventional HL3	*Mix 3: SP19 3% RAS 25% RAP*
Mean	7.25	8.88
Variance	16.98	20.23
Observations	240	240
df	239	239
F	**0.84**	
P(F<=f) one-tail	8.80E-02	
F Critical one-tail	**0.81**	

From Figure 5-19, it can be observed that Mix 2 has a slow rate of deterioration compared to the rest of the mixes while Mix 1 exhibited the highest rate of deterioration. Mix 2 is observed to perform better than the control mix, Conventional HL 3 over the analysis period.

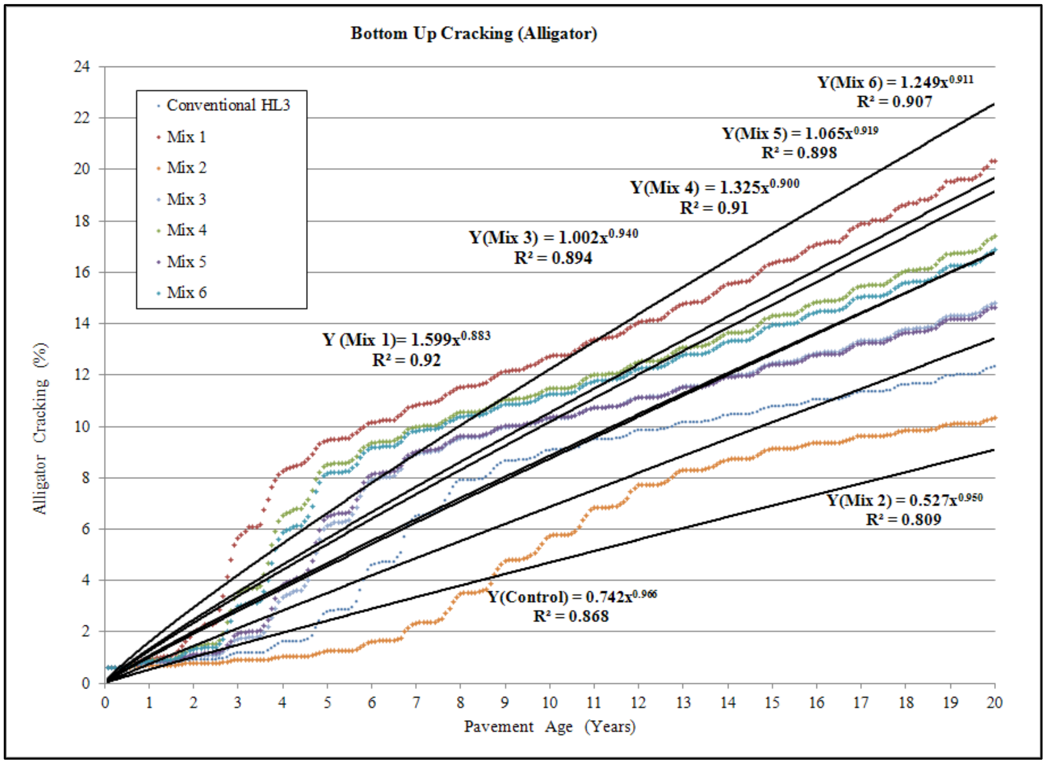


Figure 5-19: Alligator Cracking Prediction Comparison to the Control Mix

5.3.4 Prediction Model for Pavement Deformation (Rutting)

From the F-test performed on the mixes, it was observed that Mix 4, and Mix 6 had no statistical difference when compared to Mix 1. The P-Value (greater than 0.1) indicates consistency in pavement performance with Mix 1in both the Asphalt Cement (AC) layer and total pavement structure as shown in Table 5-12 and Table 5-13.

Mixes 2, 3, and 5 exhibit very strong statistical evidence of variation of pavement performance from Mix 1in both the asphalt layer and overall total pavement structure including all the asphalt, base, and subbase layers. The P-Value (less than 0.001) strongly favors the three mixes over Mix 1 in terms of resisting rutting in the pavement layers.

Table 5-12: Total Pavement Deformation Statistical Comparison to Mix 1

F-Test Two-Sample for Variances

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	15.52	11.64
Variance	9.91	4.08
Observations	240	240
df	239	239
F	**2.43**	
P(F<=f) one-tail	7.23E-12	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	15.52	13.54
Variance	9.91	6.66
Observations	240	240
df	239	239
F	**1.49**	
P(F<=f) one-tail	0.001	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	15.52	14.99
Variance	9.91	9.12
Observations	240	240
df	239	239
F	**1.09**	
P(F<=f) one-tail	0.260	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	15.52	13.81
Variance	9.91	6.76
Observations	240	240
df	239	239
F	**1.46**	
P(F<=f) one-tail	0.002	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	15.52	14.91
Variance	9.91	9.13
Observations	240	240
df	239	239
F	**1.09**	
P(F<=f) one-tail	0.264	
F Critical one-tail	**1.24**	

Table 5-13: Asphalt Cement (AC) Deformation Statistical Comparison to Mix 1

F-Test Two-Sample for Variances

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	5.30	3.00
Variance	3.69	1.15
Observations	240	240
df	239	239
F	**3.22**	
P(F<=f) one-tail	6.99E-19	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	5.30	4.05
Variance	3.69	2.19
Observations	240	240
df	239	239
F	**1.69**	
P(F<=f) one-tail	3.01E-05	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	5.30	5.07
Variance	3.69	3.43
Observations	240	240
df	239	239
F	**1.08**	
P(F<=f) one-tail	0.28	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	5.30	4.31
Variance	3.69	2.33
Observations	240	240
df	239	239
F	**1.58**	
P(F<=f) one-tail	2.00E-04	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	5.30	5.06
Variance	3.69	3.45
Observations	240	240
df	239	239
F	**1.07**	
P(F<=f) one-tail	0.30	
F Critical one-tail	**1.24**	

The rate of deformation in the pavement consisting of alternative mixes is slightly slower than Mix 1in both the total pavement structure and asphalt cement layer as shown in Figure 5-20 and Figure 5-21 respectively. The difference in the rate of deformation in the pavement structure was approximately 13.6% while for the asphalt cement layer it was 9.4%. This percentage difference can greatly affect the project cost over the service life.

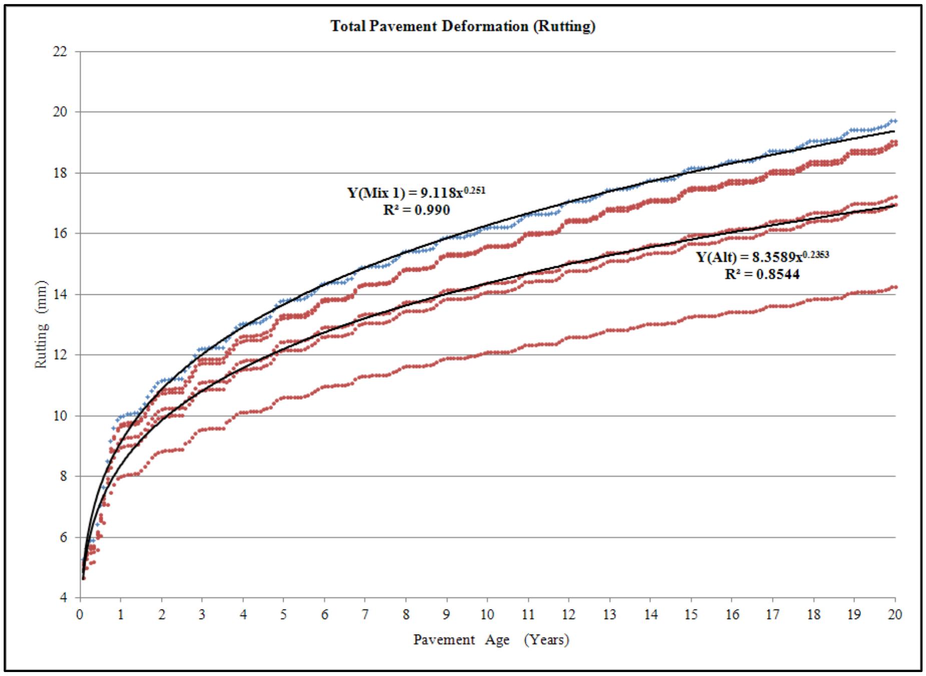


Figure 5-20: Total Pavement Deformation Prediction Comparison to Mix 1

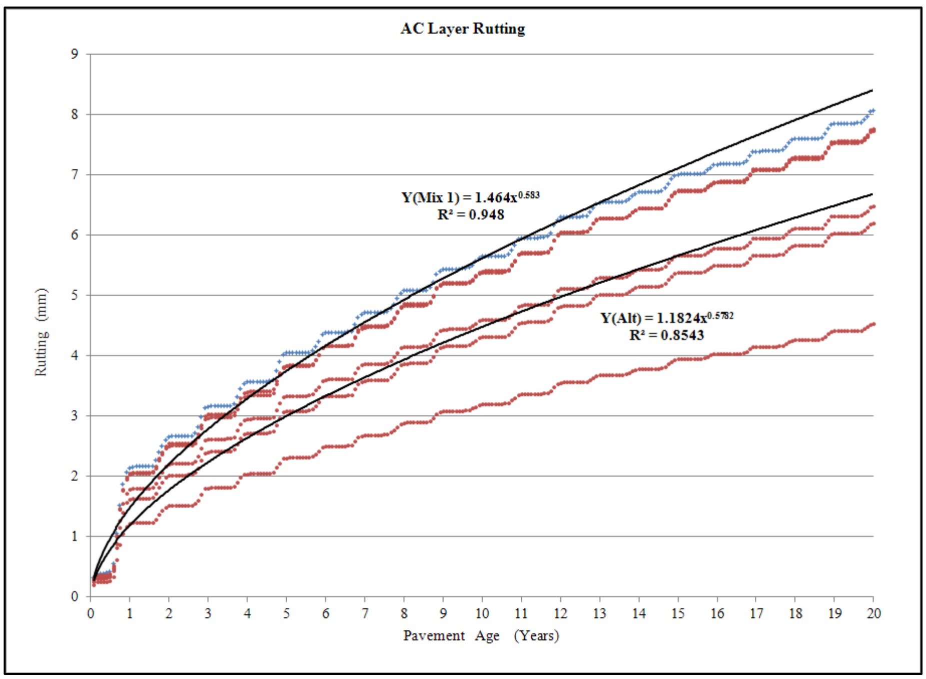


Figure 5-21: Asphalt Cement (AC) Layer Deformation Prediction Comparison to Mix 1

Comparison to the control, Conventional HL 3 had a consistent pattern of deformation with Mixes 1, 4, and 6 as illustrated by the statistical analysis given in Table 5-14 and Table 5-15. However, Mix 3 and Mix 5 show less statistical variation while Mix 2 shows strong statistical variation as compared to the control in both asphalt cement layer and total pavement deformation. Mix 2 exhibits better resistance to rutting than the control and the rest of the mixes as illustrated in Figure 5-22. This would be related to the fact that the RAS stiffens the mix to better resist rutting. The R-value also indicates that the designed mixes with RAS and/or RAP will perform in a similar way if not better than the control. This is encouraging as one of the concerns with using RAS and/or RAP is to ensure they perform in similar or better than the control mix.

Table 5-14: AC Deformation Statistical Comparison to the Control Mix

F-Test Two-Sample for Variances: AC Deformation

	Conventional HL3	*Mix 1: HL3 1.5% RAS 13.5% RAP*
Mean	3.83	5.30
Variance	1.96	3.69
Observations	240	240
df	239	239
F	**0.53**	
P(F<=f) one-tail	6.56E-07	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	3.83	5.07
Variance	1.96	3.43
Observations	240	240
df	239	239
F	**0.57**	
P(F<=f) one-tail	9.54E-06	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 5: SP12.5 FC2 6% RAS*
Mean	3.83	4.31
Variance	1.96	2.33
Observations	240	240
df	239	239
F	**0.84**	
P(F<=f) one-tail	0.093	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	3.83	5.06
Variance	1.96	3.45
Observations	240	240
df	239	239
F	**0.57**	
P(F<=f) one-tail	0.000	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 2: SP19 6% RAS*
Mean	3.83	3.00
Variance	1.96	1.15
Observations	240	240
df	239	239
F	**1.71**	
P(F<=f) one-tail	0.000	
F Critical one-tail	**1.24**	

	Conventional HL3	*Mix 3: SP19 3% RAS 25% RAP*
Mean	3.83	4.05
Variance	1.96	2.19
Observations	240	240
df	239	239
F	**0.90**	
P(F<=f) one-tail	0.200	
F Critical one-tail	**0.81**	

Table 5-15: Total Pavement Deformation Statistical Comparison to the Control Mix

F-Test Two-Sample for Variances: Total Pavement Deformation

	Conventional HL3	*Mix 1: HL3 1.5% RAS 13.5% RAP*
Mean	12.87	15.52
Variance	5.92	9.91
Observations	240	240
df	239	239
F	**0.60**	
P(F<=f) one-tail	3.81E-05	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	12.87	14.99
Variance	5.92	9.12
Observations	240	240
df	239	239
F	**0.65**	
P(F<=f) one-tail	4.46E-04	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 5: SP12.5 FC2 6% RAS*
Mean	12.87	13.81
Variance	5.92	6.76
Observations	240	240
df	239	239
F	**0.88**	
P(F<=f) one-tail	0.152	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	12.87	14.91
Variance	5.92	9.13
Observations	240	240
df	239	239
F	**0.65**	
P(F<=f) one-tail	0.000	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 2: SP19 6% RAS*
Mean	12.87	11.64
Variance	5.92	4.08
Observations	240	240
df	239	239
F	**1.45**	
P(F<=f) one-tail	0.002	
F Critical one-tail	**1.24**	

	Conventional HL3	*Mix 3: SP19 3% RAS 25% RAP*
Mean	12.87	13.54
Variance	5.92	6.66
Observations	240	240
df	239	239
F	**0.89**	
P(F<=f) one-tail	0.182	
F Critical one-tail	**0.81**	

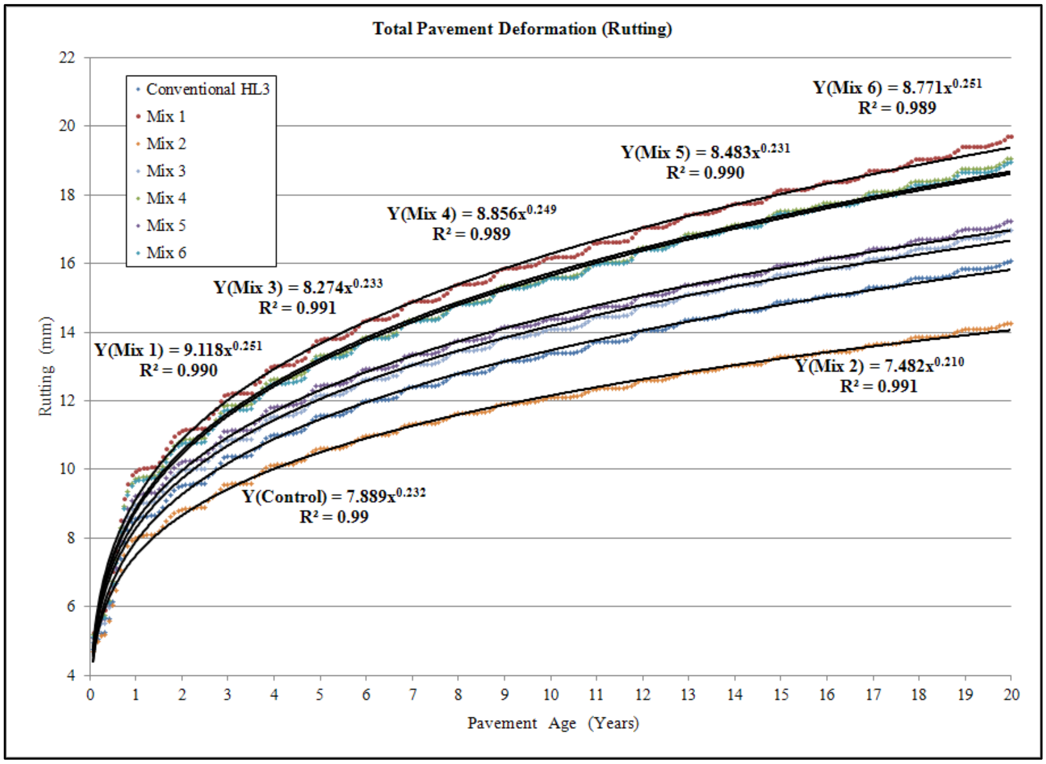


Figure 5-22: Total Pavement Deformation Prediction Comparison to the Control Mix

5.3.5 Prediction Model for International Roughness Index (IRI)

The F-test indicated that Mixes 4 and 6 had no statistical variation in the rate of deterioration from Mix 1 hence the two mixes exhibit consistent properties with Mix 1 (P-Value > 0.1). Mixes 2, 3, and 5 had statistical variation in the rate of deterioration against Mix 1 as illustrated in Table 5-16.

Mix 2 showed very strong statistical evidence of variation in the rate at which roughness occurs to the pavement surface (P-Value < 0.001) while Mixes 3 and 5 illustrated moderate (0.01 < P-Value < 0.05) evidence of variation from Mix 1.

The HMA mixes incorporated with the recycled materials exhibited a linear rate of deterioration as shown in Figure 5-23. There was a 4% difference in the rate of deterioration between Mix 1 and the five alternative mixes.

Table 5-16: IRI Statistical Comparison to Mix 1

F-Test Two-Sample for Variances

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	1.82	1.66
Variance	0.07	0.04
Observations	240	240
df	239	239
F	**1.63**	
P(F<=f) one-tail	8.08E-05	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	1.82	1.74
Variance	0.07	0.06
Observations	240	240
df	239	239
F	**1.27**	
P(F<=f) one-tail	0.03	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	1.82	1.79
Variance	0.07	0.07
Observations	240	240
df	239	239
F	**1.10**	
P(F<=f) one-tail	0.24	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	1.82	1.75
Variance	0.07	0.06
Observations	240	240
df	239	239
F	**1.28**	
P(F<=f) one-tail	0.03	
F Critical one-tail	**1.24**	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	1.82	1.79
Variance	0.07	0.06
Observations	240	240
df	239	239
F	**1.11**	
P(F<=f) one-tail	0.21	
F Critical one-tail	**1.24**	

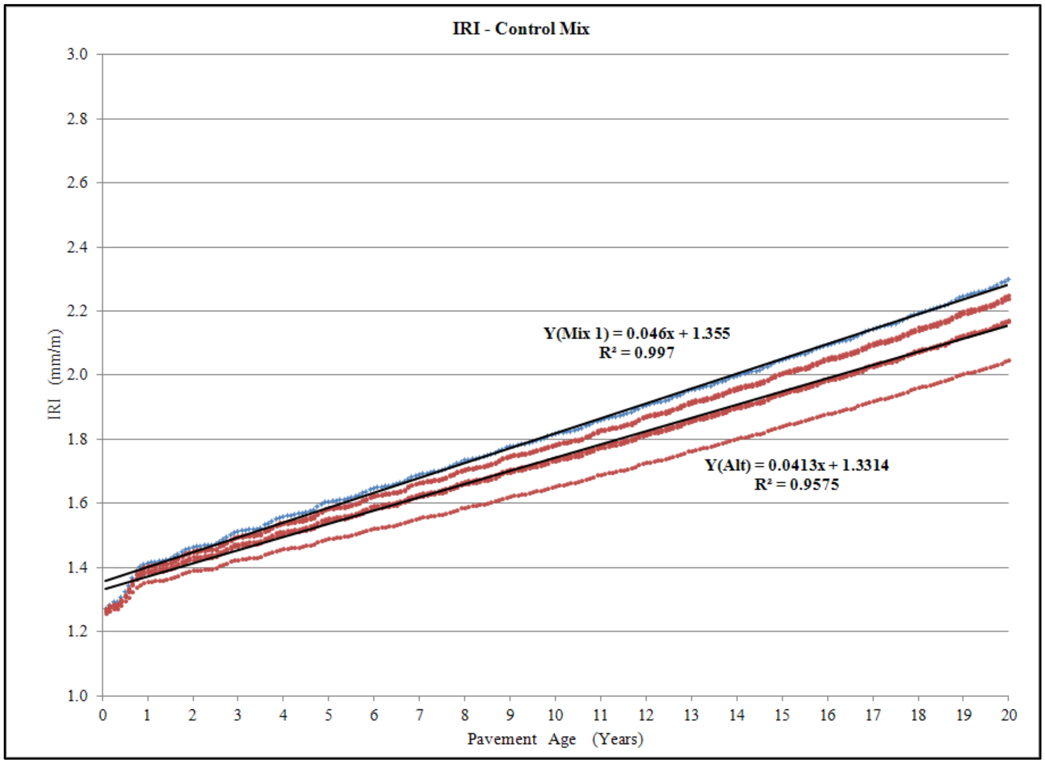


Figure 5-23: IRI Prediction Comparison to Mix 1

Comparison with conventional HL 3 illustrated some inconsistencies in statistical data as illustrated in Table 5-17. Mix 3 and Mix 5 were observed to have a significant statistically IRI difference whereas the other mixes had similar rate of deterioration with conventional HL 3. However the R-value (R^2 = 99%) indicated no significant differences between the mixes with RAS and/or RAP and conventional HL 3 demonstrated in Figure 5-24.

Table 5-17: IRI Statistical Comparison to the Control Mix

F-Test Two-Sample for Variances

	Conventional HL3	*Mix 1: HL3 1.5% RAS 13.5% RAP*
Mean	1.71	1.82
Variance	0.05	0.07
Observations	240	240
df	239	239
F	**0.71**	
P(F<=f) one-tail	0.005	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	1.71	1.79
Variance	0.05	0.07
Observations	240	240
df	239	239
F	**0.78**	
P(F<=f) one-tail	0.029	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 5: SP12.5 FC2 6% RAS*
Mean	1.71	1.75
Variance	0.05	0.06
Observations	240	240
df	239	239
F	**0.91**	
P(F<=f) one-tail	0.239	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	1.71	1.79
Variance	0.05	0.06
Observations	240	240
df	239	239
F	**0.79**	
P(F<=f) one-tail	0.036	
F Critical one-tail	**0.81**	

	Conventional HL3	*Mix 2: SP19 6% RAS*
Mean	1.71	1.66
Variance	0.05	0.04
Observations	240	240
df	239	239
F	**1.17**	
P(F<=f) one-tail	0.118	
F Critical one-tail	**1.24**	

	Conventional HL3	*Mix 3: SP19 3% RAS 25% RAP*
Mean	1.71	1.74
Variance	0.05	0.06
Observations	240	240
df	239	239
F	**0.91**	
P(F<=f) one-tail	0.226	
F Critical one-tail	**0.81**	

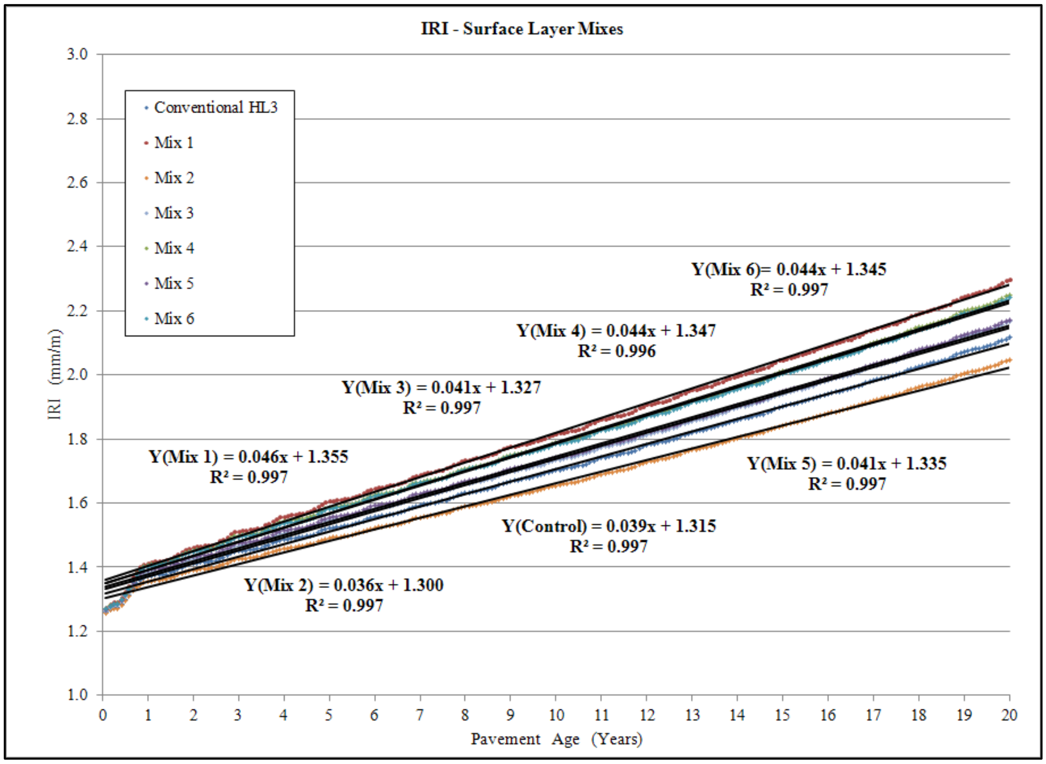


Figure 5-24: IRI Prediction Comparison to Control Mix

5.4 Summary

Based on MEPDG modeling, mixes without the RAP content demonstrated slightly better performance over the service life compared to mixes with RAP as shown by predictions from pavement distresses and damage analysis models. Performance analyses indicated that there would be minimal bottom-up damage/cracking (alligator), roughness (IRI), and rutting in both the AC layer and total pavement structure. This should be revisited as field performance in Ontario has shown equal or better performance of RAP mixes when compared to mixes without RAP.

All mixes performed well in rutting over the design period except Mix 1, which reached its terminal serviceability value. However; with preventive maintenance, the pavements with the designed mixes with RAS and/or RAP promise to perform similarly if not better than the control, Conventional HL 3. Therefore the mixes are anticipated to adequately support the designed traffic load under natural climatic condition.

Overall, Mix 2 (SP 19 6%), binder layer exhibited better performance in all distress resistance analysis followed by surface layer Mix 5 (SP 12.5 6%). Mix 3 (SP 19 3% RAS, 25% RAP), Mix 4 (SP 12.5 3% RAS, 17% RAP) and Mix 6 (SP 12.5 3% RAS, 12% RAP) exhibited similar performance properties to Mix 1: HL 3 1.5% RAS, 13.5% RAP. However, it should be noted that from an environmental perspective it is very desirable to use RAP in combination with RAS.

Chapter 6: Life-Cycle Assessment (LCA) of RAS Pavement

6.1 Introduction

This chapter presents the life cycle stages associated with the HMA mixtures incorporating with RAS and/or RAP. Life-Cycle Assessment is a standardized comprehensive methodology which can be used for analyzing and quantifying the environmental impacts, economic effects, and sustainability of a product and/or process [Greenroads^{TM} 2010]. The framework evaluates the environmental and economic performance of the pavement construction projects at the project level by analysing the initial construction and maintenance of pavement life cycle stages according to the pavement structural layers for the material production, transportation, and process [Nathman 2008].

The life cycle assessment is divided into two parts (1) Environmental Impacts Assessment using PaLATE and (2) Economic Assessment using LCCA methodology. The assessment quantified the sustainability of the pavement materials under the study by evaluating the savings incurred without compromising on pavement performance and escalated project costs. The LCA estimated the environmental and economic savings as a series of pavement treatments totalled over the analysis period of the pavement. The environmental savings estimated emissions and energy emitted by the different HMA mixtures at the various life-cycle stages whereas economic savings analyzed pavement material savings and treatments over the design life. LCA evaluates the stages of the product's life including raw material extraction, transportation, processing, usage and disposal. Four steps are involved in this evaluation, namely: (1) Goal Definition and Scope, (2) Inventory Analysis, (3) Impact Assessment and (4) Interpretation [Austin 2011].

6.2 Quantification of Environmental Savings using PaLATE

6.2.1 Introduction

The PaLATE framework shown on Figure 6-1 was used in this assessment to evaluate the performance of the pavement during its life cycle [Horvath 2003]. The framework uses life cycle assessment to model the environmental effects of pavement construction as well as maintenance by defining the pavement design criteria. For this research, only the initial construction of the wearing course was analyzed. The pavement design then yielded the type and volume of construction material needed by defining a combination of construction activities. The analysis steps are given below:

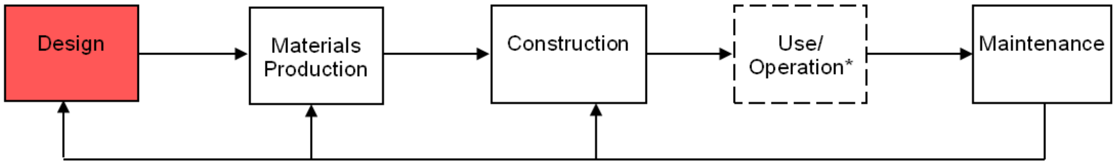
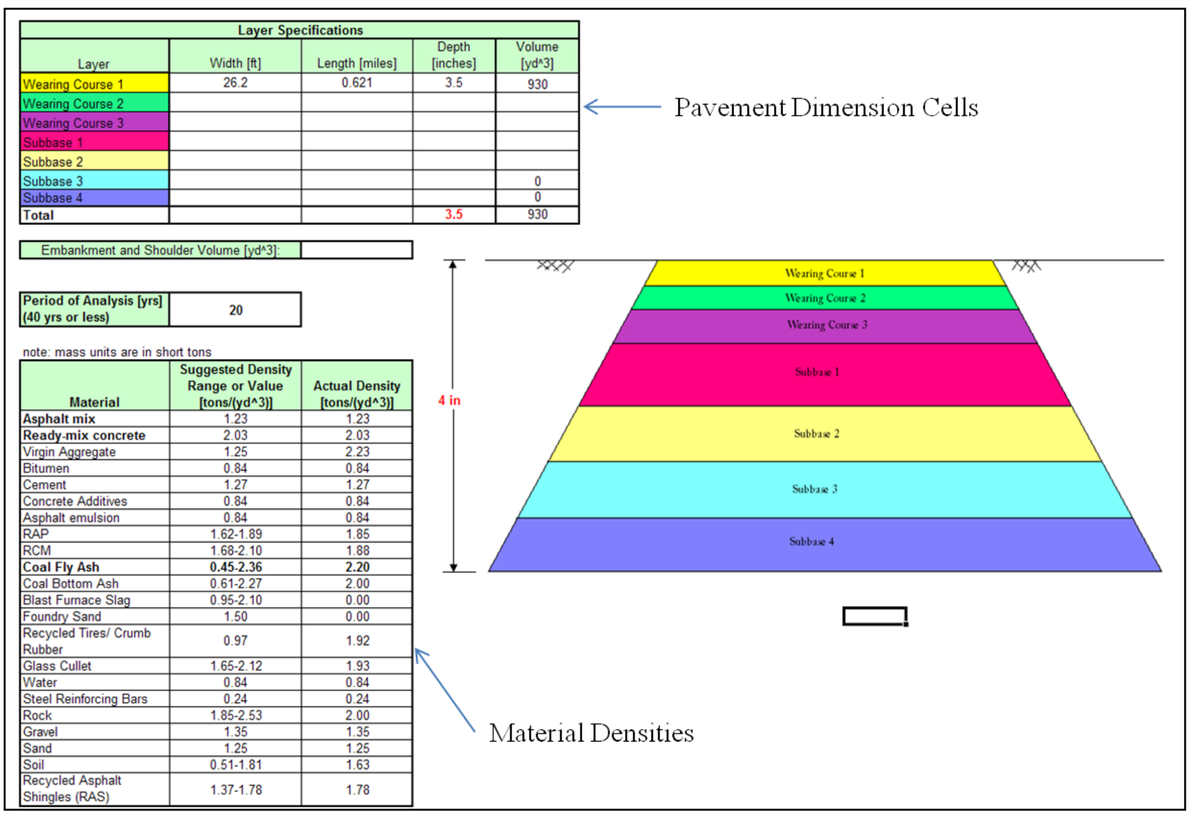


Figure 6-1: Life Cycle Pavement Phases

Step 1: Design sheet is where the dimensions of pavement layers, density of the construction materials, and the period of analysis were defined. The period of analysis was used for discounting purposes of the economic analysis. The volume of the layers, in this case only one layer was used as the comparison is amongst various mixes combined with material density calculated the mass of each material used. Figure 6-2 illustrates the design sheet and the pavement layers.



Layer Specifications				
Layer	Width [ft]	Length [miles]	Depth [inches]	Volume [yd^3]
Wearing Course 1	26.2	0.621	3.5	930
Wearing Course 2				
Wearing Course 3				
Subbase 1				
Subbase 2				
Subbase 3				0
Subbase 4				0
Total			3.5	930

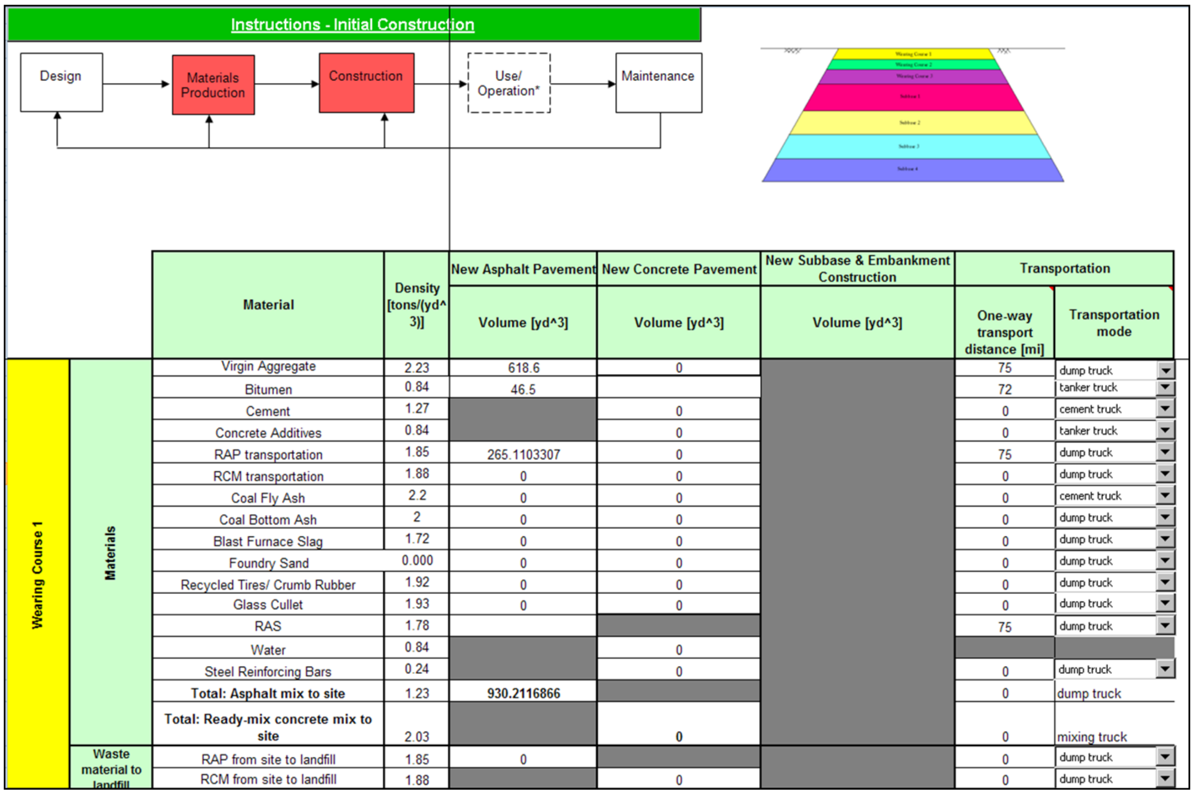
Embankment and Shoulder Volume [yd^3]:	
Period of Analysis [yrs] (40 yrs or less)	20

note: mass units are in short tons

Material	Suggested Density Range or Value [tons/(yd^3)]	Actual Density [tons/(yd^3)]
Asphalt mix	1.23	1.23
Ready-mix concrete	2.03	2.03
Virgin Aggregate	1.25	2.23
Bitumen	0.84	0.84
Cement	1.27	1.27
Concrete Additives	0.84	0.84
Asphalt emulsion	0.84	0.84
RAP	1.62-1.89	1.85
RCM	1.68-2.10	1.88
Coal Fly Ash	0.45-2.36	2.20
Coal Bottom Ash	0.61-2.27	2.00
Blast Furnace Slag	0.95-2.10	0.00
Foundry Sand	1.50	0.00
Recycled Tires/ Crumb Rubber	0.97	1.92
Glass Cullet	1.65-2.12	1.93
Water	0.84	0.84
Steel Reinforcing Bars	0.24	0.24
Rock	1.85-2.53	2.00
Gravel	1.35	1.35
Sand	1.25	1.25
Soil	0.51-1.81	1.63
Recycled Asphalt Shingles (RAS)	1.37-1.78	1.78

Figure 6-2: Design Worksheet

Step 2: Initial Construction Worksheet, is where the inputs for pavement material volumes, transportation distances, and method of material transportation in the different pavement layers were defined such as materials for wearing course 1 – 3, subbase 1 – 4, and embankment and shoulder materials. For this research, wearing course 1 was used in the analysis as shown in Figure 6-3. The maintenance worksheet designed to incorporate pavement maintenance and rehabilitation processes such as Hot In-place Recycling (HIR), Cold In-place Recycling (CIR), and Full Depth Reclamation (FDR) is similar to the initial construction worksheet, however, it was not used for the study.



		Material	Density [tons/(yd^3)]	New Asphalt Pavement Volume [yd^3]	New Concrete Pavement Volume [yd^3]	New Subbase & Embankment Construction Volume [yd^3]	Transportation: One-way transport distance [mi]	Transportation: Transportation mode
Wearing Course 1	Materials	Virgin Aggregate	2.23	618.6	0		75	dump truck
		Bitumen	0.84	46.5			72	tanker truck
		Cement	1.27		0		0	cement truck
		Concrete Additives	0.84		0		0	tanker truck
		RAP transportation	1.85	265.1103307	0		75	dump truck
		RCM transportation	1.88	0	0		0	dump truck
		Coal Fly Ash	2.2	0	0		0	cement truck
		Coal Bottom Ash	2	0	0		0	dump truck
		Blast Furnace Slag	1.72	0	0		0	dump truck
		Foundry Sand	0.000	0	0		0	dump truck
		Recycled Tires/ Crumb Rubber	1.92	0	0		0	dump truck
		Glass Cullet	1.93	0	0		0	dump truck
		RAS	1.78				75	dump truck
		Water	0.84		0			
		Steel Reinforcing Bars	0.24		0		0	dump truck
		Total: Asphalt mix to site	1.23	**930.2116866**			0	dump truck
		Total: Ready-mix concrete mix to site	2.03		**0**		0	mixing truck
	Waste material to landfill	RAP from site to landfill	1.85	0			0	dump truck
		RCM from site to landfill	1.88		0		0	dump truck

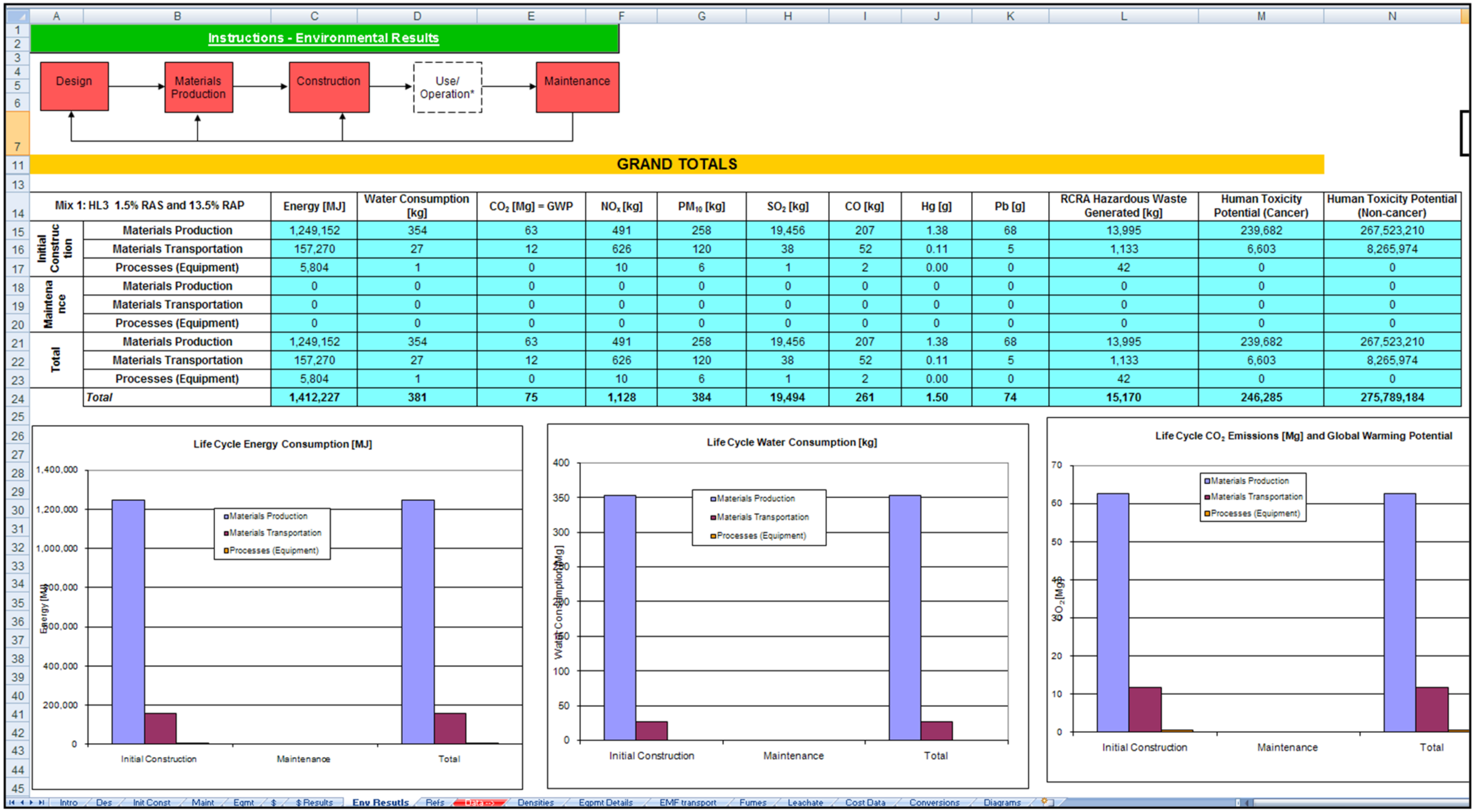
Figure 6-3: Initial Construction Worksheet

Step 3: Environmental Results Worksheet, reports environmental effects resulting from the initial construction and maintenance phases, and by material production, transport and processing. Hence the worksheet summarizes all the life cycle assessment quantities shown in both numerical and bar charts explaining emissions let into the environment. Energy use and emissions are based on typical productivity, fuel consumption rate, and engine size of the equipment used in each activity. Environmental effects depend on the characteristics of equipment used to recover the construction material and the hauling distances of the material between processing facilities and construction site. No inputs were required for this worksheet and it was locked to avoid any accidental incidences.

The environmental impacts estimated by PaLATE included water and energy usage, global warming potential (CO_2), pollutant emissions, RCRA hazardous waste release, human toxicity potential, fumes and leachate which are summarized in Table 6-1 while PaLATE analysis results shown in Figure 6-4. The PaLATE process output considers the materials and equipment activities used during the initial pavement construction.

Table 6-1: PaLATE Estimated Environmental Results

Environmental Results	Measurement Units
Energy	Mega-joule (MJ)
Water Consumption	Kilogram (kg)
CO_2	Mega-gram (Mg)
NO_x	Kilogram (kg)
PM_{10}	Kilogram (kg)
SO_2	Kilogram (kg)
CO	Kilogram (kg)
Hg	Gram (g)
Pb	Gram (g)
RCRA Hazardous Waste Generated	Kilogram (kg)
Human Toxicity Potential (Cancer)	HTP
Human Toxicity Potential (Non-Cancer)	HTP



Mix 1: HL3 1.5% RAS and 13.5% RAP		Energy [MJ]	Water Consumption [kg]	CO_2 [Mg] = GWP	NO_x [kg]	PM_{10} [kg]	SO_2 [kg]	CO [kg]	Hg [g]	Pb [g]	RCRA Hazardous Waste Generated [kg]	Human Toxicity Potential (Cancer)	Human Toxicity Potential (Non-cancer)
Initial Construction	Materials Production	1,249,152	354	63	491	258	19,456	207	1.38	68	13,995	239,682	267,523,210
	Materials Transportation	157,270	27	12	626	120	38	52	0.11	5	1,133	6,603	8,265,974
	Processes (Equipment)	5,804	1	0	10	6	1	2	0.00	0	42	0	0
Maintenance	Materials Production	0	0	0	0	0	0	0	0	0	0	0	0
	Materials Transportation	0	0	0	0	0	0	0	0	0	0	0	0
	Processes (Equipment)	0	0	0	0	0	0	0	0	0	0	0	0
Total	Materials Production	1,249,152	354	63	491	258	19,456	207	1.38	68	13,995	239,682	267,523,210
	Materials Transportation	157,270	27	12	626	120	38	52	0.11	5	1,133	6,603	8,265,974
	Processes (Equipment)	5,804	1	0	10	6	1	2	0.00	0	42	0	0
	Total	**1,412,227**	**381**	**75**	**1,128**	**384**	**19,494**	**261**	**1.50**	**74**	**15,170**	**246,285**	**275,789,184**

Figure 6-4: Environmental Results Worksheet

6.2.2 PaLATE User Input Interface for Construction

The PaLATE tool uses the user inputs for the design, initial construction, maintenance, and equipment used to characterise the life cycle environmental effects of the given project (HMA mixtures). The tool uses an alternative approach for assessing the impacts from the pavement construction and maintenance based on the productivity and environmental impacts caused by the different equipment and material types. PaLATE also uses the EIO-LCA (Environmental Input-Output Life Cycle Analysis) database that calculates emissions stemming from the material production. The tool also uses EPA data and information to quantify any environmental penalty of constructing and maintaining pavements by roughly estimating trade-offs between virgin and recycled materials [Horvath 2004]. Using PaLATE the goal was to evaluate the environmental effects of using the different quantities of RAS and/or RAP in HMA pavements.

The first step was to define the pavement layer dimensions, construction material and analysis period as illustrated above in Figure 6-2. The quantities of the materials required for the analysis were calculated using the CPATT Test Track pavement structure geometry as shown in Table 6-2. PaLATE requires the user to convert every quantity to volume.

For the purpose of determining environmental effects in this thesis, it was assumed that RAP only contributes to total volume of aggregates required for the HMA. Past research has shown that RAP provides approximately 5-7% asphalt binder [Epps 1977].

PaLATE does not calculate environmental effects due to RAS, therefore from previous research, it is reported that shingles contain approximately 30% to 40% asphalt cement by weight. Past research reported that total amount of asphalt binder is decreased with incorporation of RAS. A 5% RAS contributes 1.5% and 3% RAS contributes 0.9% by weight to total asphalt binder [Austin 2011]. The quantity of RAS was added to RAP and then analysed in PaLATE.

First Step is to calculate the total pavement asphalt required using Equation 6-1

$$\text{TPA} = \text{Width (ft)} \times \text{Length (miles)} \times \text{Depth (inches)} \qquad \text{Equation 6-1}$$

$$\text{TPA} = \mathbf{930yd^3}$$

Table 6-2: User Input for Layer Specification

Layer Specifications				
Layer	Width [ft]	Length [miles]	Depth [inches]	Volume [yd^3]
Wearing Course 1	26.2	0.621	3.5	930
Wearing Course 2				
Wearing Course 3				
Subbase 1				
Subbase 2				
Subbase 3				0
Subbase 4				0
Total			**3.5**	930

Second step is to determine the quantities of virgin aggregate, asphalt binder, and RAP. This was determined using the HMA mix designs provided by Miller Paving Ltd.

For Conventional HL 3

Assuming 5% asphalt binder by weight and air voids of 4% by volume: asphalt binder volume required for analysis was calculated by determining the total quantity of asphalt mix using Equation 6-2.

$$W_{Mix} = Total\ Pavement\ Asphalt\ x\ HMA\ Mix\ Density$$

Equation 6-2

$$W_{mix} = 930yd^3 \text{ x } 2.16tons/yd^3 = \mathbf{2008.8tons}$$

Total asphalt binder is 5% by weight; the binder weight is calculated from Equation 6-3.

$$W_{Binder} = Asphalt\ Cement\ Percent\ x\ W_{Mix}$$

Equation 6-3

$$W_{Binder} = 0.05 \text{ x } 2008.8tons = \mathbf{100.44tons}$$

The total volume of asphalt binder required can be calculated using the binder density assumed to be 0.84tons/yd^3 in Equation 6-4.

$$V_{Binder} = \left(Weight\ of\ binder\ (W_{Binder}) \middle/ Density\ of\ Binder \right)$$

Equation 6-4

$$V_{Binder} = \frac{100.44tons}{0.84tons/yd^3} = \mathbf{119.5714yd^3}$$

Assuming 88% by volume of the entire asphalt mixture is made of aggregates, volume of aggregate was calculated using Equation 6-5.

$$V_{Aggregate} = 930yd^3\ x\ 0.88 = \mathbf{818.4yd^3}$$

Equation 6-5

All HMA mixes incorporated with RAS and/or RAP, the total volumes in the mixture were determined using their material densities and proportions designed by Miller Paving Ltd using Equation 6-6.

$$V_{RAS\ or\ RAP} = \left(\frac{Total\ Volume\ per\ layer\ x\ Density\ of\ HMA}{Density\ of\ RAS\ or\ RAP}\right) x\ Material\ Percentage$$

Equation 6-5

***<u>For example:</u>* HL 3: 1.5% RAS and 13.5% RAP**

$$V_{RAP} = \frac{2008.8tons\ x\ 2.16tons/yd^3\ x\ 0.135}{1.85tons/yd^3} = \mathbf{316.63yd^3}$$

$$V_{RAS} = \frac{2008.8tons\ x\ 2.16tons/yd^3\ x\ 0.015}{1.7tons/yd^3} = \mathbf{38.28yd^3}$$

The volume of RAP is then subtracted from the total volume of virgin aggregates to determine the required virgin aggregate while the volume of virgin binder is determined after subtracting the binder from RAS. Table 6-3 summarizes the PaLATE inputs.

Table 6-3: PaLATE Inputs in Metric

Mixture	Asphalt Binder (m^3)	Aggregates (m^3)	RAP (m^3)	Total HMA (m^3)	Distance (km)
Control: Conventional HL 3	91.4	653.8	0	711	120
Mix 1: HL 3 (1.5% RAS + 13.5% RAP)	72.5	563.1	118.9	711	120
Mix 2: SP 19 (6% RAS)	59.9	625.7	52.4	711	120
Mix 3: SP 19 (3% RAS + 25% RAP)	52.8	486.2	221.7	711	120
Mix 4: SP 12.5 FC 1 (3% RAS + 17% RAP)	59.7	536.0	158.5	711	120
Mix 5: SP 12.5 FC 2 (6% RAS)	65.4	623.8	52.2	711	120
Mix 6: SP 12.5 FC 2 (3% RAS + 12% RAP)	67.1	565.9	119.1	711	120

Note: 1cubic yard (yd^3) is equivalent to 0.764m^3

6.2.3 PaLATE Results

For this thesis, only the construction stage was analysed. Results from PaLATE are given in a graphical interface as shown in Figure 6-5 to Figure 6-7 for the HMA mix GHG emissions, energy usage and water usage. Only energy usage, water usage, and CO_2 are represented graphically for all the design mixes as these have a direct effect on the environment, the rest of the results are found in Appendix B. CO_2 output reflects global warming potential. Table 6-4summarises the results for all the mixes using PaLATE.

To evaluate the savings two control mixes were used, Control Mix: Conventional HL 3 and Mix 1: HL 3 1.5% RAS and 13.5% RAP. Mix 1 was used to analyse the effect of increase in the percentage of RAS in HMA while Control Mix was for analysing the sustainability of using recycled material in HMA. According to PaLATE results; Mix 3: SP19 3% RAS and 25% RAP had least quantity of emissions into the environment as well as consuming less water and energy.

Table 6-4: PaLATE HMA Outputs for Initial Construction

Description	Grande Totals						
	Conventional HL 3	**HL 3 (1.5% RAS + 13.5% RAP)**	**SP 19 (6% RAS)**	**SP 19 (3% RAS + 25% RAP)**	**SP 12.5 FC 1 (3% RAS + 17% RAP)**	**SP 12.5 FC 2 (6% RAS)**	**SP 12.5 FC 2 (3% RAS + 12% RAP)**
Energy (MJ)	2,722,558	2,278,819	2,025,968	1,821,160	1,990,567	2,145,808	2,161,325
Water Consumption (kg)	903	725	614	541	607	664	676
CO_2 (Kg) = GWP	148,817	123,551	109,517	97,541	107,238	116,251	116,939
NO_x (kg)	1,556	1,431	1,343	1,298	1,344	1,381	1,393
PM_{10} (kg)	535	473	488	417	447	493	469
SO_2 (kg)	20,782	20,914	20,392	20,955	20,826	20,517	20,815
CO (kg)	560	460	396	355	393	425	432
Hg (kg)	0	0	0	0	0	0	0
Pb (kg)	0	0	0	0	0	0	0
RCRA Hazardous Waste Generated (kg)	36,834	29,531	24,691	21,906	24,603	26,813	27,439
Human Toxicity Potential (Cancer)	580,842	468,448	393,464	351,036	392,464	426,243	436,160
Human Toxicity Potential (Non-cancer)	361,211,409	319,797,525	349,355,143	284,830,036	307,662,051	348,288,282	321,315,604

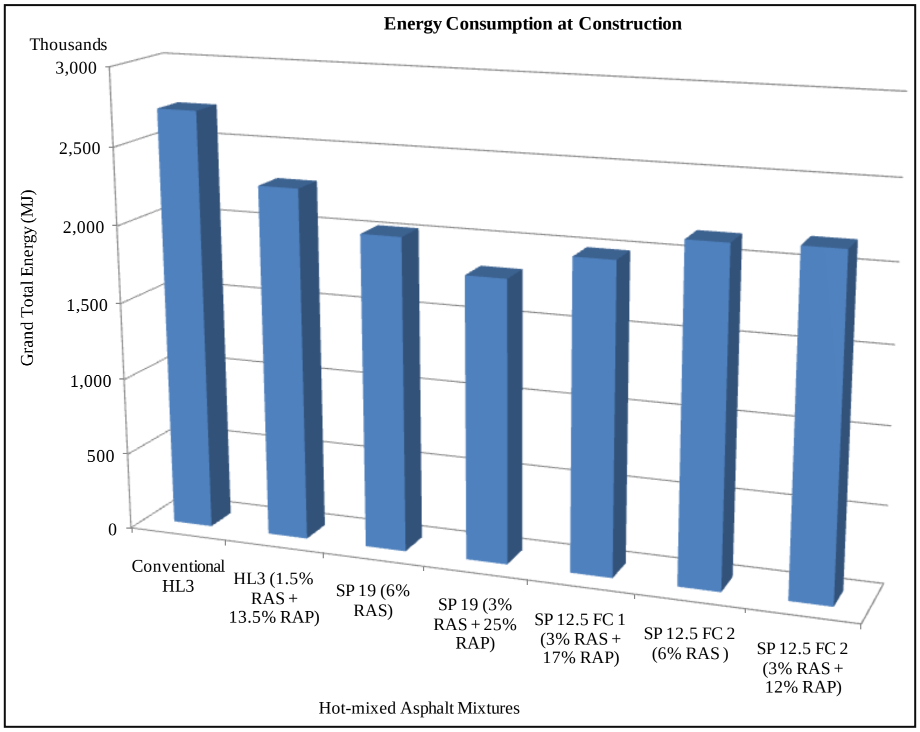


Figure 6-5: Energy Consumption Output for Construction

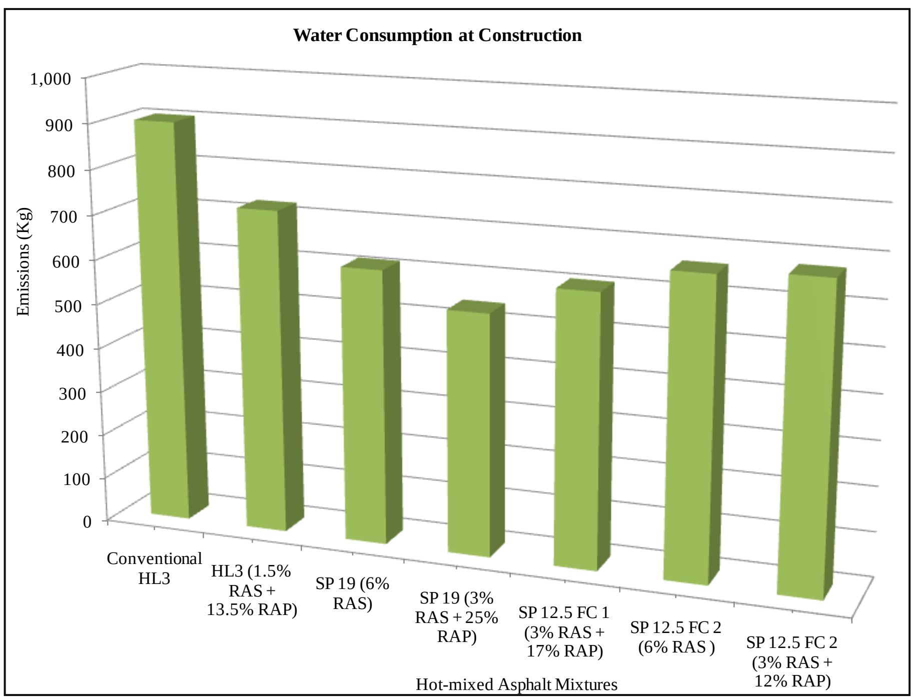


Figure 6-6: Water Consumption Output for Construction

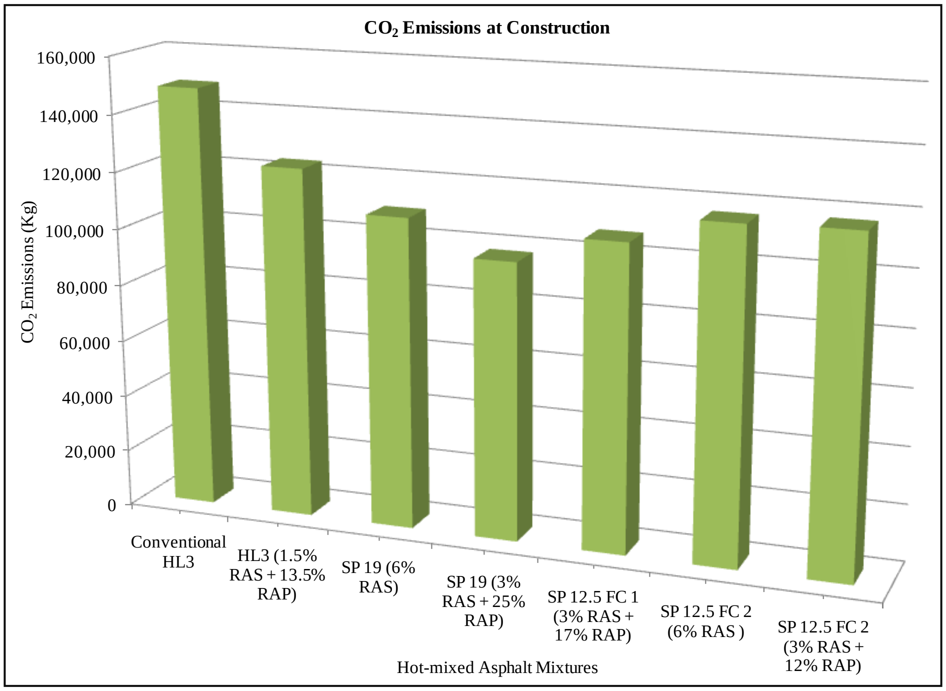


Figure 6-7: CO_2 Emission at Construction

6.2.4 Environmental Savings

A comparison between the control mixes and the alternative mixes of the environmental emissions, energy and water consumption incurred by the HMA mixtures incorporated with recycled materials was formulated using Equation 6-7 to determine the relative percentage savings.

$$RPS = \left({(Control\ Mix - Alternative\ Mix)}/{Control\ Mix} \right) * 100$$

Equation 6-7

Where; RPS – Relative Percentage Savings

Control Mix – Conventional HL 3

Mix 1 – HL 3 1.5% RAS and 13.5% RAP

Alternative Mix –Five mixes with RAS and/or RAP

The results attained by the Equation 6-7 are shown in Table 6-5. It was observed in both Figure 6-8 and Figure 6-9 that Mix 3: SP19 3% RAS, 25% RAP had the highest average percentage savings of

18% followed by Mix 4: SP12.5 FC1 3% RAS, 17% RAP (11.3%) when compared to Mix 1. However, it was observed that mixes with 6% RAS incorporation did not have any significant savings in Particular Matter (PM_{10}) as shown in Figure 6-9, in fact there was an increase in emission of PM_{10} compared to Mix 1. Higher savings in Particular Matter (PM_{10}) were observed for other mixes. This is beneficial for health purposes as inhaling PM_{10} can lead to harmful respiratory effects with time. It was also observed that mixes with 6% RAS exhibited negative savings in human toxicity potential although the type of toxicity was non-cancerous but had higher savings in SO_2 compared to other mixes.

Therefore from PaLATE analysis, Mix 3: SP19 3% RAS, 25% RAP is considered the optimal sustainable option for use in HMA pavement when compared to Mix 1.

Table 6-5: Relative Percentage Savings in Comparison with Mix 1

Environmental Emission	**Relative Environmental Percentage Savings (%)**				
	SP 19 (6% RAS)	**SP 19 (3% RAS + 25% RAP)**	**SP 12.5 FC 1 (3% RAS + 17% RAP)**	**SP 12.5 FC 2 (6% RAS)**	**SP 12.5 FC 2 (3% RAS + 12% RAP)**
Energy (MJ)	11.1	20.1	12.6	5.8	5.2
Water Consumption (kg)	15.3	25.4	16.3	8.4	6.8
CO_2 (Kg) = GWP	11.4	21.1	13.2	5.9	5.4
NO_x (kg)	6.1	9.3	6.1	3.5	2.6
PM_{10} (kg)	-3.1	11.9	5.6	-4.2	1.0
SO_2 (kg)	2.5	-0.2	0.4	1.9	0.5
CO (kg)	13.8	22.8	14.6	7.6	6.1
Hg (kg)	16.7	25.9	16.8	9.4	7.2
Pb (kg)	15.6	25.5	16.4	8.7	6.9
RCRA Hazardous Waste Generated (kg)	16.4	25.8	16.7	9.2	7.1
Human Toxicity Potential (Cancer)	16.0	25.1	16.2	9.0	6.9
Human Toxicity Potential (Non-cancer)	-9.2	10.9	3.8	-8.9	-0.5
Average Savings	**9.9**	**18.0**	**11.3**	**5.2**	**4.6**

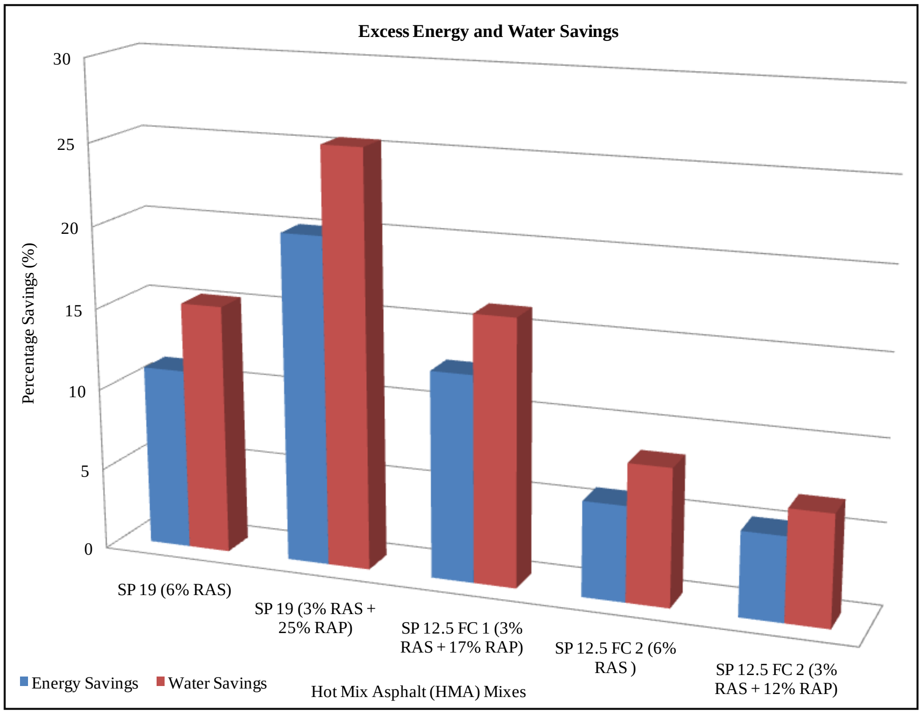


Figure 6-8: Excess Energy and Water Savings

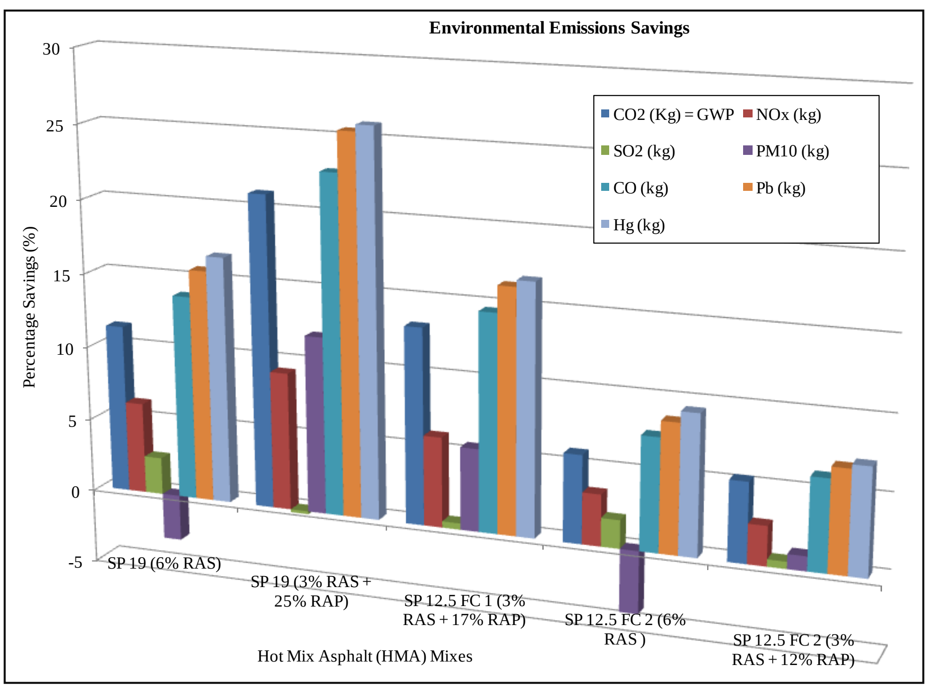


Figure 6-9: Environmental Emissions Savings

6.2.5 Comparison with Conventional HL 3

All the engineered HMA mixes with RAS and/or RAP were compared and validated with conventional HL 3 to assess the mix with the highest relative savings given in Table 6-6. However, it should be noted that having the highest savings does not mean that it is the optimal mix, other factors influencing the HMA mix have to be considered such as structural characteristics, economic effects, workability and source of the material. An optimal sustainable mix should be one that does not compromise pavement performance or possess financial constraints.

Mix 3: SP19 3% RAS, 25% RAP was observed to have the highest average environmental and consumption savings (29.2%) followed by Mix 4: SP12.5 FC1 3% RAS, 17% RAP (23.7%) as illustrated by Figure 6-10 and Figure 6-11. A negative saving in SO_2 was observed in all mixes except for mixes with 6% RAS.

Table 6-6: Relative Percentage Savings in Comparison with Conventional HL 3

Environmental Emission	Relative Environmental Percentage Savings (%)					
	HL 3 (1.5% RAS + 13.5% RAP)	**SP 19 (6% RAS)**	**SP 19 (3% RAS + 25% RAP)**	**SP 12.5 FC 1 (3% RAS + 17% RAP)**	**SP 12.5 FC 2 (6% RAS)**	**SP 12.5 FC 2 (3% RAS + 12% RAP)**
Energy (MJ)	16.3	25.6	33.1	26.9	21.2	20.6
Water Consumption (kg)	19.7	32.0	40.1	32.7	26.4	25.1
CO2 (Kg) = GWP	17.0	26.4	34.5	27.9	21.9	21.4
NOx (kg)	8.0	13.7	16.6	13.6	11.2	10.5
PM10 (kg)	11.5	8.8	22.1	16.5	7.8	12.4
SO2 (kg)	-0.6	1.9	-0.8	-0.2	1.3	-0.2
CO (kg)	18.0	29.3	36.7	29.9	24.2	23.0
Hg (kg)	19.9	33.2	40.7	33.3	27.4	25.6
Pb (kg)	19.7	32.2	40.2	32.8	26.6	25.2
RCRA Hazardous Waste Generated (kg)	19.8	33.0	40.5	33.2	27.2	25.5
Human Toxicity Potential (Cancer)	19.4	32.3	39.6	32.4	26.6	24.9
Human Toxicity Potential (Non-cancer)	11.5	3.3	21.1	14.8	3.6	11.0
Average Savings	**14.4**	**22.6**	**29.2**	**23.7**	**18.7**	**18.2**

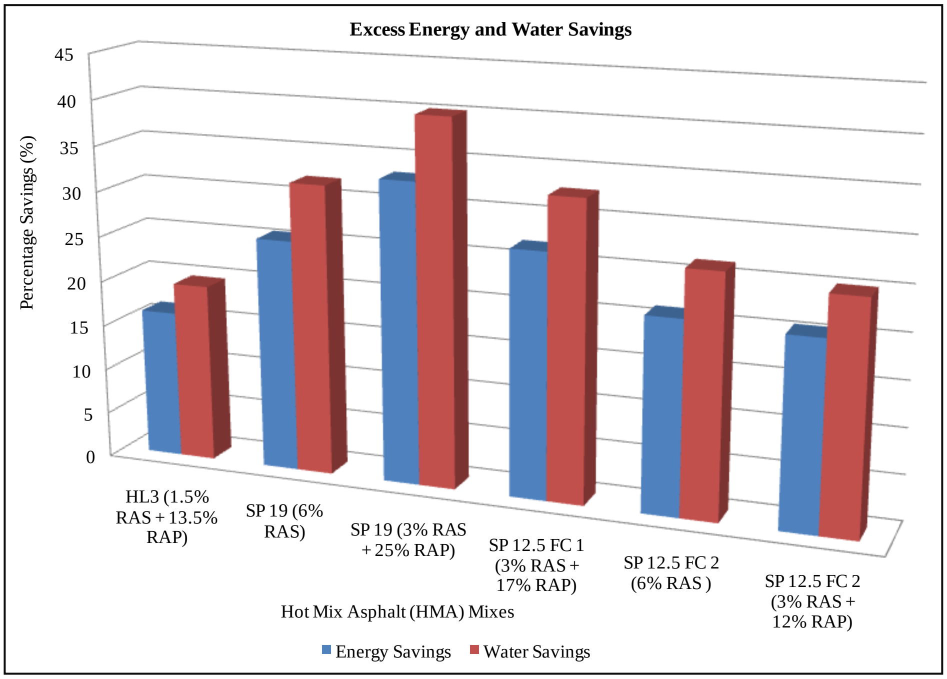


Figure 6-10: Excess Energy and Water Relative Savings

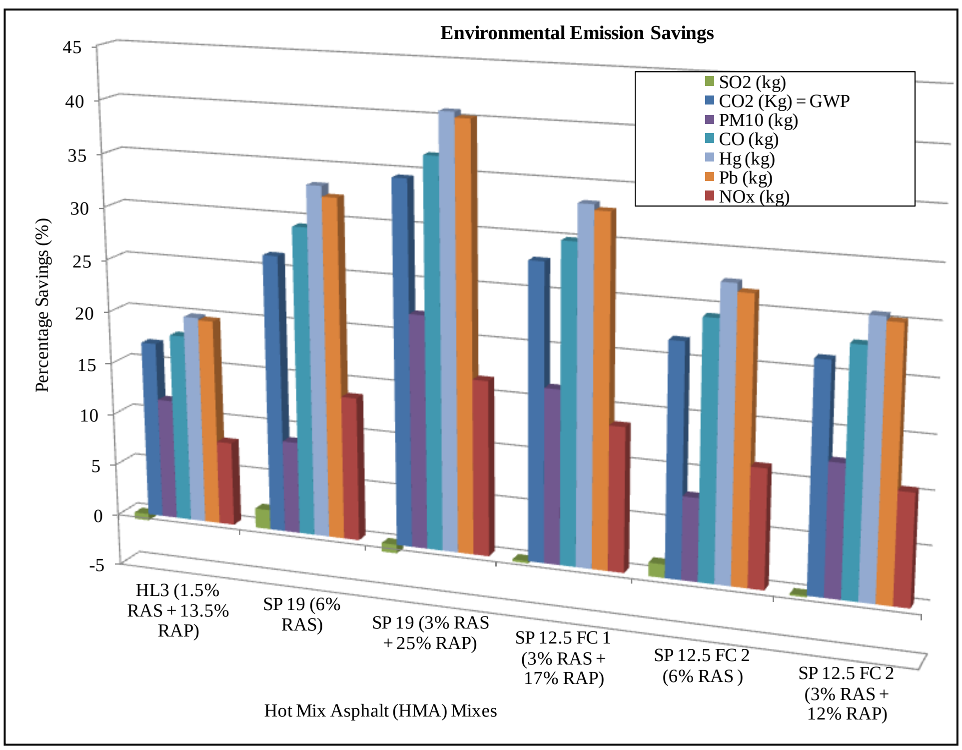


Figure 6-11: Environmental Emission Relative Savings

6.3 Quantification of Economic Savings Using Life-Cycle Cost Analysis (LCCA)

6.3.1 Introduction

The quantification of economic saving relates life cycle costs with pavement performance over the life cycle stages. This is attained through a technique that uses economic principles to compare the competing alternative strategies for maintenance and rehabilitation (M&R) of pavements, which is a subset of Benefit-Cost Analysis (BCA).

The assumptions for LCCA performed on the control mixes and five alternative mixes was for a traffic loading of 2500 Annual Average Daily Truck Traffic (AADTT). Using the MTO recommendations and maintenance reports prepared on the 50-year design analysis, Maintenance and Rehabilitation (M&R) schedules were designed for all the mixes under the study. Initial costs for constructing a flexible pavement were based on the MTO life-cycle cost analysis reports [ARA 2011], [ARA 2007], and [Smith 1998]. From the structural analysis, pavement distresses were observed for the different mixes and the year of maintenance and rehabilitation table was formulated as shown in Table 6-7. This indicated when major maintenance and rehabilitation will be needed depending on the severity and extent of the distresses.

For the maintenance and rehabilitation activities; rout and seal, pavement resurfacing, mill and patch activities were chosen for pavements carrying a traffic load of 2500 AADTT in this study as shown in Table 6-8 to Table 6-10. The MTO unit costs carried out over a 25-year study for the particular traffic load were used in the analysis of life cycle analysis. From the cost spreadsheets, initial construction cost was calculated; maintenance/rehabilitation schedules made and cost annualized over the analysis period.

Table 6-7: Pavement Distress and Condition Criteria used as Triggers for M&R

Pavement Surface Distresses	Assessment Level	Conventional HL 3	Maintenance and Rehabilitation Needs Year: HL 3 1.5% RAS 13.5% RAP	SP19 6% RAS	SP19 3% RAS 25% RAP	SP12.5 FC1 3% RAS 17% RAP	SP12.5 FC2 6% RAS	SP12.5 FC2 3% RAS 12% RAP
IRI	1.6mm/m	7.0	5.0	8.5	6.5	5.8	6.3	5.8
	2mm/m	17.5	14.0	19.0	16.5	14.8	16.5	14.8
Rutting - Total Pave	13mm	8.8	4.0	14.0	7.0	4.8	6.5	4.8
	19mm	N/A	18.0	N/A	N/A	20.0	N/A	20.0
Rutting - AC	4mm	9.8	4.9	15.0	8.8	5.8	7.8	5.8
Alligator Cracking	10%	12.8	6.0	19.0	9.0	7.0	9.0	7.5
	20%	N/A	19.8	N/A	N/A	N/A	N/A	N/A
Alligator Damage	10%	20.0	9.5	N/A	15.0	12.0	15.0	12.0
	20%	N/A	18.0	N/A	N/A	N/A	N/A	N/A
Longitudinal Cracking	1000m/km	3.8	3.0	4.1	3.6	3.5	3.5	3.5
Longitudinal Damage	20%	4.2	3.3	4.5	3.6	3.5	3.5	3.5

**N/A (Not Applicable) meaning no treatment was required as the particular distress was not observed in the pavement.

Table 6-8: Maintenance and Rehabilitation Program for Control Mix and Mix 1

Mix Description	Proposed Maintenance and Rehabilitation Activity	Year
Control Mix: Conventional HMA	Rout and Crack Sealing (200 m/km)	3
	5% Mill and patch 40 mm	8
	20% Mill and patch 40 mm	15
	20% Mill and Patch 40mm	20
Mix 1: HL 3 1.5% RAS, 13.5% RAP	Rout and Crack Sealing (200 m/km)	3
	5% Mill and patch 40 mm	5
	20% Mill and patch 40 mm	9
	20% Mill and Patch 40mm	15
	Mill 90mm/Place 90mm Asphalt Pavement (Overlay)	19

Table 6-9: Maintenance and Rehabilitation Program for Surface Layer Mixes

Mix Description	Proposed Maintenance and Rehabilitation Activity	Year
Mix 4: SP 12.5 FC1 3% RAS, 17% RAP	Rout and Crack Sealing (200 m/km)	3
	5% Mill and patch 40 mm	6
	20% Mill and Patch 40mm	10
	20% Mill and Patch 40mm	15
	SuperPave 12.5mm,FC1 40mm	20
Mix 5: SP 12.5 FC2 6% RAS	Rout and Crack Sealing (200 m/km)	3
	5% Mill and patch 40 mm	6
	20% Mill and Patch 40mm	10
	20% Mill and Patch 40mm	15
	SuperPave 12.5mm,FC1 40mm	20
Mix 6: SP 12.5 FC2 3% RAS, 12% RAP	Rout and Crack Sealing (200 m/km)	3
	5% Mill and patch 40 mm	6
	20% Mill and Patch 40mm	9
	20% Mill and Patch 40mm	15

Table 6-10: Maintenance and Rehabilitation Program for Binder Layer Mixes

Mix Description	Proposed Maintenance and Rehabilitation Activity	Year
Mix 2: SP 19 6% RAS	Rout and Crack Sealing (200 m/km)	4
	5% Mill and patch 40 mm	9
	20% Mill and Patch 40mm	15
	20% Mill and Patch 40mm	19
Mix 3: SP 19 3% RAS, 25% RAP	Rout and Crack Sealing (200 m/km)	3
	5% Mill and patch 40 mm	6
	20% Mill and Patch 40mm	9
	20% Mill and Patch 40mm	15

6.3.2 Initial Construction Costing

Using MTO LCCA reports, initial construction estimates per mix were calculated as shown in Table 6-11 to Table 6-13 [Smith 1998], [ARA 2007]. It was observed that constructing HMA pavement incorporated with RAS and RAP was more expensive compared to when only RAS was used alone for both surface and binder layers. The initial costs are the ones that are assumed to occur in the base year (year zero) of the analysis period of the project.

Table 6-11: Initial Construction Cost for Control Mix and Mix 1

Conventional HMA

Pavement Layer	Description of Pavement Layer	Amount	Quantity per Km	Unit Cost	Total Cost
Surface	Conventional HMA	40	1,512	$ 58.61	$ 88,618.32
Binder	SP 19, mm (t)	110	4,059	$ 96.00	$ 389,664.00
Base	Granular A, mm (t)	150	5,400	$ 18.00	$ 97,200.00
Subbase	Granular B, mm (t)	450	13,500	$ 15.00	$ 202,500.00
Excavation	Earth Excavation (m^3)	750	11,250	$ 18.00	$ 202,500.00
Grand Total Initial Construction Cost					**$ 980,482.32**

HL 3 1.5% RAS 13.5% RAP

Pavement Layer	Description of Pavement Layer	Amount	Quantity per Km	Unit Cost	Total Cost
Surface	HL 3 1.5% RAS, 13.5% RAP	40	1,512	$ 49.47	$ 74,798.64
Binder	SP 19, mm (t)	110	4,059	$ 96.00	$ 389,664.00
Base	Granular A, mm (t)	150	5,400	$ 18.00	$ 97,200.00
Subbase	Granular B, mm (t)	450	13,500	$ 15.00	$ 202,500.00
Excavation	Earth Excavation (m^3)	750	11,250	$ 18.00	$ 202,500.00
Grand Total Initial Construction Cost					**$ 966,662.64**

Table 6-12: Initial Construction Costs for Surface Mixes

SP 12.5 FC1 3% RAS, 17% RAP

Pavement Layer	Description of Pavement Layer	Amount	Quantity per Km	Unit Cost	Total Cost
Surface	*SP 12.5 FC1 3% RAS, 17% RAP*	40	1,512	$ 55.37	$ 83,719.44
Binder	SP 19, mm (t)	110	4,059	$ 96.00	$ 389,664.00
Base	Granular A, mm (t)	150	5,400	$ 18.00	$ 97,200.00
Subbase	Granular B, mm (t)	450	13,500	$ 15.00	$ 202,500.00
Excavation	Earth Excavation (m^3)	750	11,250	$ 18.00	$ 202,500.00
Grand Total Initial Construction Cost					**$ 975,583.44**

SP 12.5 FC2 6% RAS

Pavement Layer	Description of Pavement Layer	Amount	Quantity per Km	Unit Cost	Total Cost
Surface	*SP 12.5 FC2 6% RAS*	40	1,512	$ 75.04	$ 113,460.48
Binder	SP 19, mm (t)	110	4,059	$ 96.00	$ 389,664.00
Base	Granular A, mm (t)	150	5,400	$ 18.00	$ 97,200.00
Subbase	Granular B, mm (t)	450	13,500	$ 15.00	$ 202,500.00
Excavation	Earth Excavation (m^3)	750	11,250	$ 18.00	$ 202,500.00
Grand Total Initial Construction Cost					**$ 1,005,324.48**

SP 12.5 FC2 3% RAS, 12% RAP

Pavement Layer	Description of Pavement Layer	Amount	Quantity per Km	Unit Cost	Total Cost
Surface	*SP 12.5 FC2 3% RAS, 12% RAP*	40	1,512	$ 65.72	$ 99,368.64
Binder	SP 19, mm (t)	110	4,059	$ 96.00	$ 389,664.00
Base	Granular A, mm (t)	150	5,400	$ 18.00	$ 97,200.00
Subbase	Granular B, mm (t)	450	13,500	$ 15.00	$ 202,500.00
Excavation	Earth Excavation (m^3)	750	11,250	$ 18.00	$ 202,500.00
Grand Total Initial Construction Cost					**$ 991,232.64**

Table 6-13: Initial Construction Costs for Binder Mixes

SP 19 6% RAS

Pavement Layer	Description of Pavement Layer	Amount	Quantity per Km	Unit Cost	Total Cost
Surface	SuperPave 12.5 FC1, mm (t)	40	1,512	$ 115.00	$ 173,880.00
Binder	*SP 19 6% RAS*	110	4,059	$ 59.90	$ 243,134.10
Base	Granular A, mm (t)	150	5,400	$ 18.00	$ 97,200.00
Subbase	Granular B, mm (t)	450	13,500	$ 15.00	$ 202,500.00
Excavation	Earth Excavation (m^3)	750	11,250	$ 18.00	$ 202,500.00
Grand Total Initial Construction Cost					**$ 919,214.10**

SP 19 3% RAS, 25% RAP

Pavement Layer	Description of Pavement Layer	Amount	Quantity per Km	Unit Cost	Total Cost
Surface	SuperPave 12.5 FC1, mm (t)	40	1,512	$ 115.00	$ 173,880.00
Binder	*SP 19 3% RAS, 25% RAP*	110	4,059	$ 46.24	$ 187,688.16
Base	Granular A, mm (t)	150	5,400	$ 18.00	$ 97,200.00
Subbase	Granular B, mm (t)	450	13,500	$ 15.00	$ 202,500.00
Excavation	Earth Excavation (m^3)	750	11,250	$ 18.00	$ 202,500.00
Grand Total Initial Construction Cost					**$ 863,768.16**

6.3.3 Life Cycle Cost Analysis

The analysis consisted of calculating the Present Worth Costs (PWC) for each mix design. PWC is the summation of all the future costs over the analysis period in today's dollars combining the discounted future maintenance and rehabilitation costs, and salvage value [Demos 2006]. This is limited to comparing the alternatives with equal analysis periods.

The three discount rates (3%, 5% and 7%) evaluated indicated consistency in the mix inputs as shown in Table 6-14. Mix 3: SP19 3% RAS and 25% RAP was observed to be the least expensive among the binder mixes while Mix 5: SP12.5 FC2 6% RAS was the least expensive among the surface layer mixes. However, in general, the binder mixes containing RAS and/or RAP were observed to cost less compared to conventional HL 3 whereas Mix 6: SP12.5 FC2 3% RAS and 12% RAP was the most expensive choice among all the mixes. Detailed life cycle costs summarized in Appendix C.

A discount rate of 5% and 7% were selected as feasible rates for this analysis as they were observed to produce minimal maintenance and rehabilitation costs throughout the analysis period.

Table 6-14: Present Worth Cost (PWC) for HMA Design Mixes at Different Discount Rates

Mix Description	Initial Cost	Total M&R Cost	Salvage Value	PWC at 3% Discount	Rank (Least Expensive)
Control: Conventional HL3	$ 980,482.32	$ 147,168.59	-$ 108,573.86	$ 1,019,077.06	3
Mix 1: HL3 1.5% RAS 13.5% RAP	$ 966,662.64	$ 200,298.61	-$ 107,043.53	$ 1,059,917.72	4
Mix 2: SP 19 6% RAS	$ 919,214.10	$ 148,282.46	-$ 101,789.31	$ 965,707.25	2
Mix 3: SP 19 3% RAS, 25% RAP	$ 863,768.16	$ 170,768.36	-$ 95,649.50	$ 938,887.03	1
Mix 4: SP 12.5 FC1 3% RAS, 17% RAP	$ 975,583.44	$ 214,777.89	-$ 108,031.38	$ 1,082,329.95	6
Mix 5: SP 12.5 FC2 6% RAS	$ 1,005,324.48	$ 170,768.36	-$ 111,324.76	$ 1,064,768.09	5
Mix 6: SP 12.5 FC2 3% RAS, 12% RAP	$ 991,232.64	$ 224,230.74	-$ 109,764.30	$ 1,105,699.09	7

Mix Description	Initial Cost	M&R	Salvage Value	PWC at 5% Discount	Rank (Least Expensive)
Control Mix: Conventional HL3	$ 980,482.32	$ 108,711.06	-$ 73,906.69	$ 1,015,286.69	3
Mix 1: HL3 1.5% RAS 13.5% RAP	$ 966,662.64	$ 159,656.16	-$ 72,865.00	$ 1,053,453.81	4
Mix 2: SP 19 6% RAS	$ 919,214.10	$ 109,802.55	-$ 69,288.43	$ 959,728.22	2
Mix 3: SP 19 3% RAS, 25% RAP	$ 863,768.16	$ 138,642.72	-$ 65,109.03	$ 937,301.86	1
Mix 4: SP 12.5 FC1 3% RAS, 17% RAP	$ 975,583.44	$ 166,972.66	-$ 73,537.43	$ 1,069,018.67	6
Mix 5: SP 12.5 FC2 6% RAS	$ 1,005,324.48	$ 138,642.72	-$ 75,779.24	$ 1,068,187.96	5
Mix 6: SP 12.5 FC2 3% RAS, 12% RAP	$ 991,232.64	$ 174,144.87	-$ 74,717.03	$ 1,090,660.48	7

Mix Description	Initial Cost	Total M&R Cost	Salvage Value	PWC at 7% Discount	Rank (Least Expensive)
Control: Conventional HL3	$ 980,482.32	$ 81,284.86	-$ 50,675.05	$ 1,011,092.13	3
Mix 1: HL3 1.5% RAS 13.5% RAP	$ 966,662.64	$ 128,700.29	-$ 49,960.80	$ 1,045,402.14	4
Mix 2: SP 19 6% RAS	$ 919,214.10	$ 82,131.36	-$ 47,508.48	$ 953,836.98	2
Mix 3: SP 19 3% RAS, 25% RAP	$ 863,768.16	$ 113,477.66	-$ 44,642.82	$ 932,602.99	1
Mix 4: SP 12.5 FC1 3% RAS, 17% RAP	$ 975,583.44	$ 131,375.98	-$ 50,421.86	$ 1,056,537.56	5
Mix 5: SP 12.5 FC2 6% RAS	$ 1,005,324.48	$ 113,477.66	-$ 51,958.99	$ 1,066,843.15	6
Mix 6: SP 12.5 FC2 3% RAS, 12% RAP	$ 991,232.64	$ 136,799.47	-$ 51,230.67	$ 1,076,801.44	7

6.3.4 Comparison to Control

An evaluation was carried out on all the design mixes containing RAS and/or RAP to analyze the variation from the control mix, conventional HL 3 in terms of cost over the analysis period. The end result however is not necessarily the selection of one alternative over the other but the selection of the most cost effective design strategy for a given situation and greater understanding of the factors influencing cost effectiveness [Walls III 1998].It was observed that binder mixes were less expensive by an average of negative 7 % while surface layer mixes were more expensive by positive 5.8% as shown in Table 6-15. Mix 3: SP19 3% RAS and 25% RAP was the most cost effective alternative among mixes containing RAS and/or RAP.

Table 6-15: Comparison of Present Worth Cost of Design Mixes to Conventional HL 3

Mix Description	PWC at 5% Discount	Comparison (%)
Control Mix: Conventional HL 3	$ 1,015,286.69	
Mix 1: HL 3 1.5% RAS 13.5% RAP	$ 1,053,453.81	3.6%
Mix 2: SP 19 6% RAS	$ 959,728.22	-5.8%
Mix 3: SP 19 3% RAS, 25% RAP	$ 937,301.86	-8.3%
Mix 4: SP 12.5 FC1 3% RAS, 17% RAP	$ 1,069,018.67	5.0%
Mix 5: SP 12.5 FC2 6% RAS	$ 1,068,187.96	5.0%
Mix 6: SP 12.5 FC2 3% RAS, 12% RAP	$ 1,090,660.48	6.9%

Mix Description	PWC at 7% Discount	Comparison (%)
Control: Conventional HL 3	$ 1,019,077.06	
Mix 1: HL 3 1.5% RAS 13.5% RAP	$ 1,059,917.72	3.9%
Mix 2: SP 19 6% RAS	$ 965,707.25	-5.5%
Mix 3: SP 19 3% RAS, 25% RAP	$ 938,887.03	-8.5%
Mix 4: SP 12.5 FC1 3% RAS, 17% RAP	$ 1,082,329.95	5.8%
Mix 5: SP 12.5 FC2 6% RAS	$ 1,064,768.09	4.3%
Mix 6: SP 12.5 FC2 3% RAS, 12% RAP	$ 1,105,699.09	7.8%

6.3.4.1 ***Sensitivity Analysis***

Sensitivity analysis involves evaluation of variability in the major input parameters affecting the Net Present Value (NPV) of a project varied over a range of levels such as percentage sensitivity level being assigned with equal weights. It was observed that the cost of the project increased with increase in sensitivity level while there was variability in cost with change in discount rates. Hence the design mixes containing RAS and/or RAP have repeatability characteristics and are consistent as shown in Figure 6-12 to Figure 6-14.

It was observed that project cost is sensitive to change in discount rates; at lower discount rate (3%) the cost of using Mix 5: SP12.5 FC2 6% RAS was relatively similar to using Mix 1: HL 3 1.5% RAS and 13.5% RAP while at higher discount rate (5%) the cost of using Mix 5: SP12.5 FC2 6% RAS was relatively similar to Mix 4: SP12.5 FC1 3% RAS and 17% RAP. At 7% discount rate, the cost of using Mix 5 was similar to both Mix 1 and Mix 4. This demonstrates that costs are very similar but cost effectiveness changes slightly with different discount; however the least expensive mix was consistent throughout all the rates analyzed.

However, a 20 year analysis does not provide a fair comparison between two or more alternatives since it usually includes only one rehabilitation strategy hence further analysis of 30, 50 and 70 years is encouraged in order to determine the most cost effective option with time.

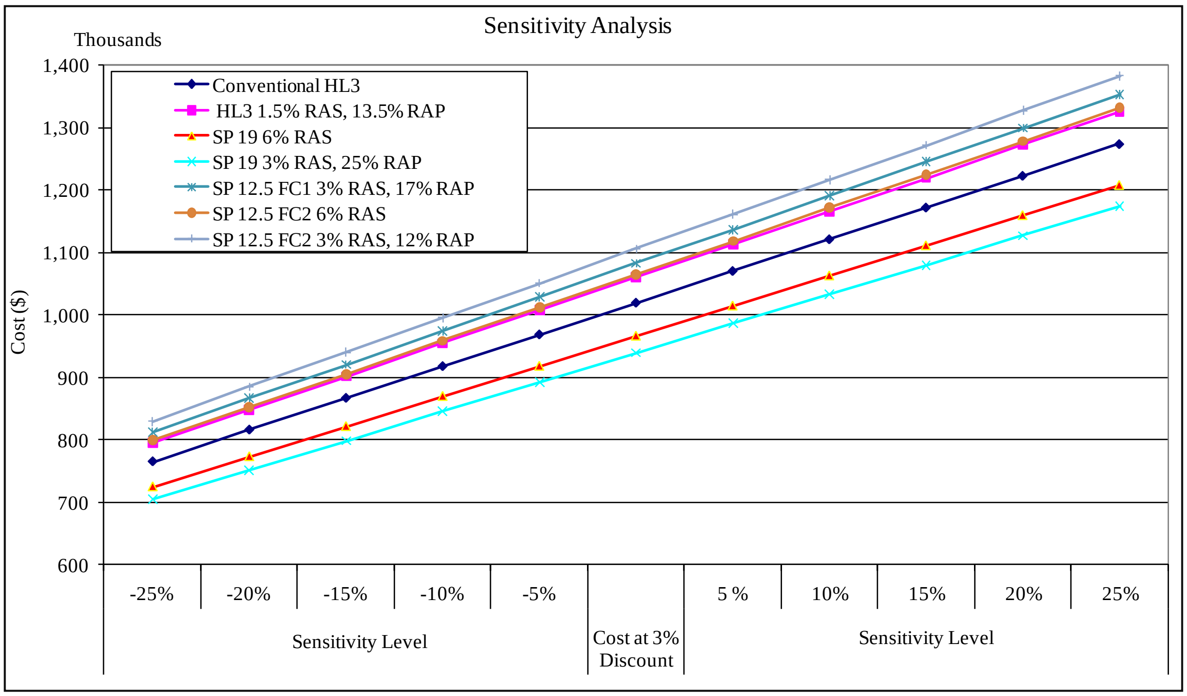


Figure 6-12: Sensitivity Analysis at 3% Discount Rate

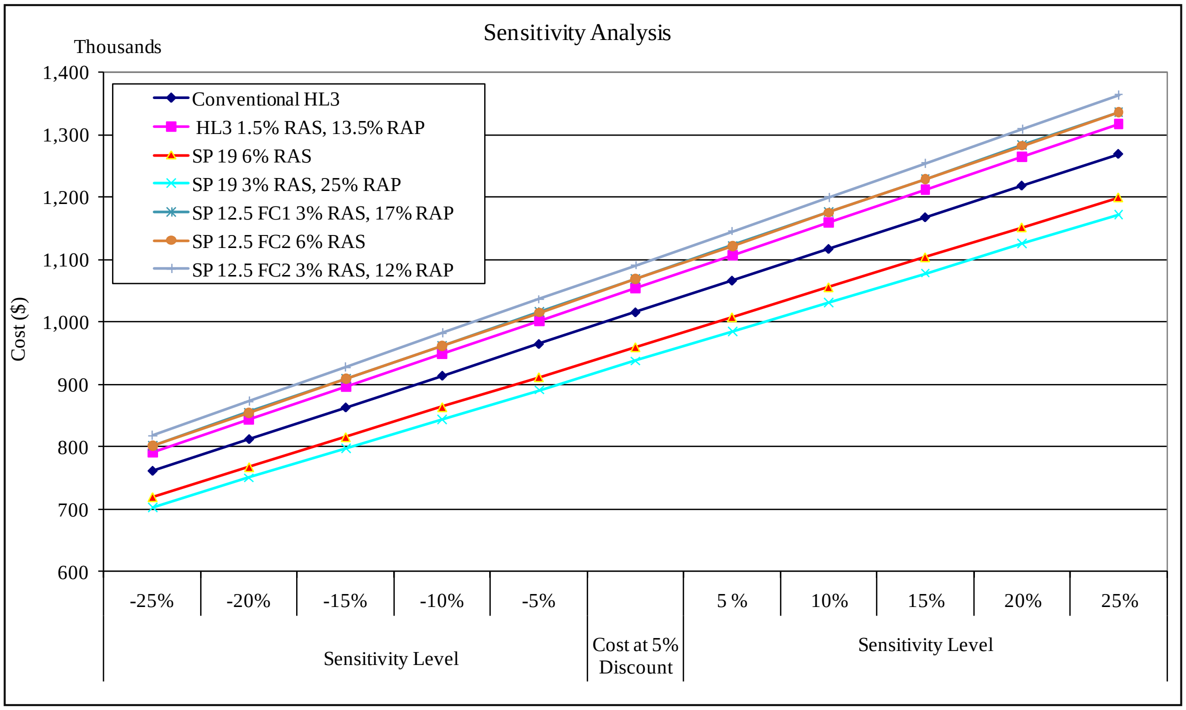


Figure 6-13: Sensitivity Analysis at 5% Discount Rate

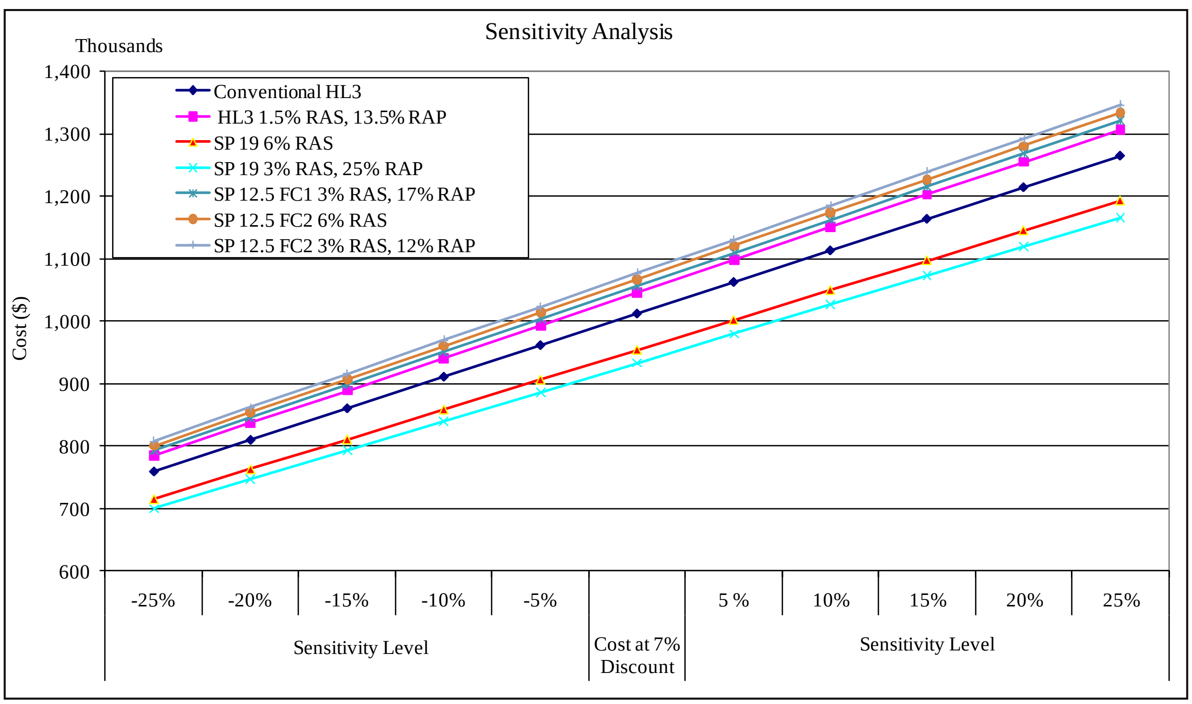


Figure 6-14: Sensitivity Analysis at 7% Discount Rate

6.4 Summary

Using life-cycle assessment (LCA), an evaluation on both environmental and economic savings was performed over a 20 year analysis period on all the mixes incorporated with RAS and/or RAP to determine the most cost effective mix. It was observed that Mix 3: SP19 3% RAS and 25% RAP demonstrated the least expensive option and exhibited higher savings due to emissions, water and energy usage. No significant variation in sensitivity analysis was observed with the different discount rate; hence it was consistent and demonstrated higher levels of repeatability.

For surface layer mixes; SP12.5 FC2 6% RAS demonstrated better cost effectiveness while Mix 4: SP12.5 FC1 3% RAS and 17% RAP had the best environmental savings and energy and water usage savings.

Chapter 7: Conclusion and Recommendations

7.1 Summary

This research involved the evaluation of six design HMA mixtures containing RAS and/or RAP through: structural analysis using the Mechanistic-Empirical Pavement Design Guide (MEPDG), Life Cycle Assessment using Pavement Life-Cycle Assessment Tool for Environmental and Economic Effects (PaLATE) for environmental assessment and Life-Cycle Cost Analysis (LCCA) for economic assessment as well as laboratory and field evaluation of pavement performance. The mixes were then validated through comparison with a control mix, a conventional HL 3 to evaluate their sustainability and performance over 20-year analysis period.

Field performance evaluation of the four test sections was an important aspect of the study. The Centre for Pavement and Transportation Technology (CPATT) Test Track, and three residential streets in the Town of Markham (Ida Street, Paul Street and Vintage Lane, and Thornhill Summit Drive) were evaluated and used to examine field performance. This is a very important aspect of the research for validating laboratory predictions. The CPATT Test Track has been in service for two years while Town of Markham residential streets have been in service for four years. The CPATT Test Track was evaluated for surface distresses, deflection and friction resistance while the residential streets were evaluated for surface distresses only. The regular distress surveys on the pavements were performed according to Ministry of Transportation Ontario (MTO) guidelines. Slabs were prepared and compacted in CPATT laboratory for each mix to simulate freeze-thaw cycles in cold climates as well as evaluate any distress manifestation for a full simulated two year analysis period.

The CPATT Test Track structural design and geometry was used for the analyses in MEPDG, PaLATE and LCCA to determine the optimal mix in terms of technical, economic and environmental costs/benefits. A traffic loading of 2500 Average Annual Daily Track Traffic (AADTT) was used in the research for all the mixes and all the mixes were designs in accordance with the Ontario Provincial Standard Specifications (OPPS). The material cost estimates were estimated using the MTO LCCA reports while the costs of Hot Mix Asphalt (HMA) containing RAS and/or RAP were obtained from Millers Paving Ltd.

One control mix was used in the study; Conventional HL 3 (Control). However, Mix 1: HL 3 1.5% RAS and 13.5% RAP also served as a primary comparison because it is well understood and has been extensively evaluated at CPATT. Mix 1 was to assess if the increase in RAS and/or RAP had an effect on HMA characteristics while the conventional HL 3 (Control) was used to evaluate the use of recycled material versus conventional HMA in pavement applications. All the analyses performed on the design mixes containing RAS and/or RAP was compared with the control, Conventional HL 3 to assess their sustainability as the control is a standard commonly used mix in Ontario.

7.2 Conclusions

Structural analysis of the pavement designs using MEPDG of the formulated six HMA mixtures containing RAS and/or RAP and Conventional HL 3 for a traffic loading of 2,500 AADDT has shown that mixes with only RAS performs slightly better than mixes with both RAS and RAP. Mix 2: SP19 6% RAS, a binder layer mix, exhibited better performance than any mixes analyzed in the study. Mix 2 performed better than the control, conventional HL 3 and exhibited less significant distresses in the 20-year analysis period. For surface layer mixes; Mix 5: SP12.5 FC2 6% RAS performed better than all surface layers mixes including Mix 1 but very similar to Conventional HL 3. However, all six mixes did not exhibit significant surface distresses such as alligator cracking/damage, roughness (IRI), and rutting in both the Asphalt Cement (AC) layer and total pavement structure. HMA mixes containing RAS and/or RAP exhibited promising good pavement performance if not better than Conventional HL 3. This finding does indicate they can provide value if incorporated into Ontario road network, especially as trade road asphalt materials become more scarce.

Life-cycle assessment was performed on all the mixes under study and it was observed that, overall; Mix 3: SP19 3% RAS and 25% RAP was the most cost effective HMA mix with the least environmental effect in terms of emissions, energy usage and water consumption. However; average environmental savings were observed to exhibit minimal variation from each other when compared to the control, Conventional HL 3.

Laboratory evaluations demonstrated good performance by the slabs prepared with mixes containing RAS. The slabs under study were still in good condition at the end of the second freeze-thaw cycle with minimal physical changes such as mass and height, which could be due to expansion and contraction during freeze thaw cycling. Mix 3: SP19 3%RAS and 25% RAP exhibited best overall performance with exceptionally high frictional resistance properties.

Field evaluations of the four test sections indicated that all the pavements with HMA containing 1.5% RAS and 13.5% RAP (CPATT Test Track, Paul Street and Vintage Lane, and Thornhill Summit Drive) were in better condition than HMA pavement containing SP12.5 FC1 3.5% RAS (Ida Street). All three of the residential streets under study and CPATT Test Track did not show significant visual distresses and the storm drains were still in good condition without any minor or major maintenance. The friction and deflection measurement tests carried out on CPATT Test Track indicated the pavement was in excellent condition despite the heavy traffic loading it carries due to the nature of its location (it exhibited excellent skid resistance measurements indicating safety for the users).

Overall, the structural analysis, life-cycle assessment, field and laboratory investigations have indicated that RAS can be a useful additive to HMA mixtures if engineered properly into the mix, in addition, cost savings can be achieved.

7.3 Recommendations

Recommendations were drawn from this study to facilitate all stakeholders involved in both public and private partnerships as well as recommendations for additional future studies on the use of RAS in HMA pavement applications.

Findings;

1. When using recycled materials such as RAS and/or RAP in HMA pavements, it is advisable to lower the Performance Graded Asphalt Binder grade by 6^0C as the final HMA mixture tends to be stiffer. By lowering the binder and making it softer, it assists in offsetting the addition of the stiff RAS. The HMA characteristics change with incorporation of RAS and/or RAP so this modification assists in achieving a good field mix.
2. Incorporation of 3% RAS or less with RAP in HMA mix does not show significant differences in pavement performance when compared to control or current HMA in Ontario. Higher percentages can be added but should be tested to ensure they can meet appropriate specifications.
3. Structural analysis using MEPDG indicated that using RAS alone in HMA performed slightly better than mixes containing both RAS and/or RAP. Mix 2: SP19 6% RAS had the best observed performance. However, this would need to be validated in the longer term through field performance. Also some further investigation into RAP performance in the MEPDG

should be investigated as in Ontario RAP HMA has been shown to perform the same or better on many roads

4. Life-Cycle Assessment (LCA) and laboratory testing indicated that Mix 3: SP19 3%RAS and 25% RAP had the best environmental savings, economic savings and best adoptability to climatic changes without loss of its safety characteristics.

Overall according to this study; Mix 3: SP19 3% RAS and 25% RAP is the optimal design mix for use in pavements in Ontario with technical, economic and environmental factors considered.

Future Work;

1. Verification of long-term performance of pavements designed with RAS in HMA mixes in terms of rutting and low-temperature cracking. The study should investigate the effect of climatic changes in the HMA pavements containing the various percentages of RAS after 5 – 10 years in service.
2. Examine the recyclability of HMA pavements containing RAS at the end of their design life.
3. Establish standard mix designs for HMA pavements containing varying percentages of RAS
4. Further comprehensive life cycle cost analysis of HMA mixes containing RAS to determine the economic viability of using RAS in pavement applications. This would examine costing as RAS becomes more readily available for usage in HMA. Currently it is available in small quantities based on demand. However, if it can be used on a more wide-spread basis, it should lead to additional savings
5. Construction of additional sections using RAS in binder layers to simulate pavement performance in medium traffic and high traffic scenarios as well as continued optimization of RAS percentages in Ontario HMA mixes with other aggregates and asphalt cement materials.
6. Further pavement predictions using MEPDG design tool for medium traffic and high traffic pavement containing RAS.
7. Continue to monitor the CPATT Test Track and three residential streets in Town of Markham to determine the in situ conditions of the pavement to determine the long-term performance. This would include taking cores at year five and year ten for laboratory evaluation.
8. Study the effect of RAP on RAS in varying HMA mixtures. Study the variability of the mixes containing only RAS to those containing both RAS and RAP.

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Appendix A: Design Mix Characteristics used in MEPDG

The new MEPDG requires that a time-temperature dependent dynamic modulus E* and poison's ratio be determined for HMA materials. The dynamic modulus used in the structural analysis is summarized in Table A-1 while the material properties of Conventional HL 3 are summarized in Table A-2.

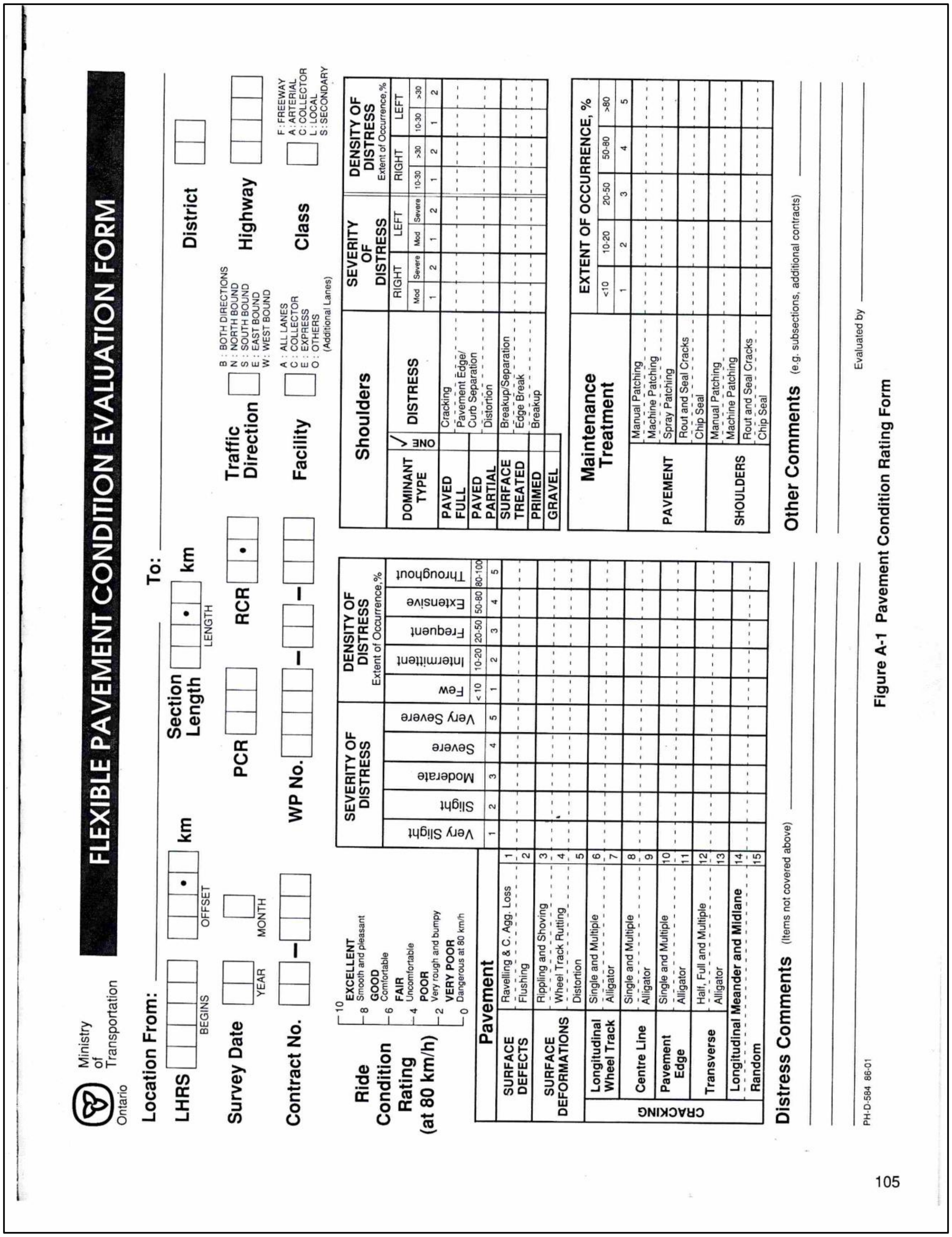
Table A-1: Dynamic Modulus (MPa) used in MEPDG

Temperature	Frequency	Conventional HL 3	HL 3 (1.5% RAS + 13.5% RAP)	SP 19 (6% RAS)	SP 19 (3% RAS + 25% RAP)	SP 12.5 FC 1 (3% RAS + 17% RAP)	SP 12.5: FC 2 (6% RAS)	SP 12.5 FC 2 (3% RAS + 12% RAP)
-10	25	2.90E+04	1.79E+04	2.80E+04	2.36E+04	2.29E+04	2.10E+04	2.40E+04
	10	2.61E+04	1.76E+04	2.71E+04	2.28E+04	2.18E+04	2.01E+04	2.27E+04
	5	2.38E+04	1.72E+04	2.62E+04	2.21E+04	2.07E+04	1.93E+04	2.09E+04
	1	1.95E+04	1.55E+04	2.35E+04	1.95E+04	1.79E+04	1.66E+04	1.85E+04
	0.5	1.70E+04	1.48E+04	2.25E+04	1.85E+04	1.67E+04	1.53E+04	1.74E+04
	0.1	1.25E+04	1.30E+04	1.99E+04	1.62E+04	1.43E+04	1.30E+04	1.46E+04
4	25	1.82E+04	1.11E+04	1.87E+04	1.60E+04	1.42E+04	1.41E+04	1.50E+04
	10	1.58E+04	1.07E+04	1.78E+04	1.51E+04	1.28E+04	1.28E+04	1.36E+04
	5	1.42E+04	1.03E+04	1.69E+04	1.38E+04	1.17E+04	1.20E+04	1.25E+04
	1	8.97E+03	8.65E+03	1.38E+04	1.14E+04	9.27E+03	9.54E+03	9.74E+03
	0.5	8.41E+03	8.03E+03	1.26E+04	1.06E+04	8.36E+03	8.69E+03	8.77E+03
	0.1	5.90E+03	6.58E+03	1.06E+04	8.62E+03	6.61E+03	6.91E+03	6.92E+03
21	25	8.52E+03	6.78E+03	1.07E+04	8.40E+03	6.94E+03	7.22E+03	7.22E+03

Temperature	Frequency	Conventional HL 3	HL 3 (1.5% RAS + 13.5% RAP)	SP 19 (6% RAS)	SP 19 (3% RAS + 25% RAP)	SP 12.5 FC 1 (3% RAS + 17% RAP)	SP 12.5: FC 2 (6% RAS)	SP 12.5 FC 2 (3% RAS + 12% RAP)
	10	6.72E+03	5.96E+03	9.73E+03	7.43E+03	6.02E+03	6.37E+03	6.29E+03
	5	5.63E+03	5.33E+03	8.90E+03	6.82E+03	5.35E+03	5.73E+03	5.55E+03
	1	3.57E+03	3.95E+03	6.91E+03	5.19E+03	4.03E+03	4.32E+03	4.13E+03
	0.5	2.90E+03	3.51E+03	6.30E+03	4.65E+03	3.66E+03	3.86E+03	3.68E+03
	0.1	1.92E+03	2.62E+03	4.90E+03	3.65E+03	2.87E+03	3.03E+03	2.85E+03
38	25	3.68E+03	3.12E+03	5.37E+03	3.80E+03	2.89E+03	3.29E+03	2.68E+03
	10	2.53E+03	2.54E+03	4.70E+03	3.17E+03	2.43E+03	2.76E+03	2.28E+03
	5	2.00E+03	2.16E+03	4.23E+03	2.76E+03	2.13E+03	2.47E+03	2.00E+03
	1	1.32E+03	1.53E+03	3.05E+03	1.99E+03	1.62E+03	1.82E+03	1.48E+03
	0.5	1.14E+03	1.36E+03	2.73E+03	1.38E+03	1.48E+03	1.65E+03	1.34E+03
	0.1	8.76E+02	1.07E+03	2.13E+03	1.42E+03	1.20E+03	1.31E+03	1.11E+03
54	25	1.77E+03	8.99E+02	1.90E+03	1.29E+03	9.31E+02	1.13E+03	9.22E+02
	10	1.24E+03	6.85E+02	1.61E+03	1.03E+03	7.76E+02	9.03E+02	7.07E+02
	5	1.06E+03	5.85E+02	1.37E+03	2.86E+03	6.74E+02	7.72E+02	6.11E+02
	1	7.89E+02	4.29E+02	9.74E+02	6.50E+02	5.27E+02	5.79E+02	4.70E+02
	0.5	7.23E+02	3.89E+02	8.78E+02	5.95E+02	4.89E+02	5.23E+02	4.35E+02
	0.1	5.43E+02	3.24E+02	6.82E+02	4.87E+02	4.26E+02	4.31E+02	3.72E+02

Table A-2: Material Properties of Conventional HL 3 [Uzarowski 2006]

Gradation	
Sieve Sizes	**Percent Passing**
25mm	
19mm	100
12.5mm	96
9.5mm	86
4.75mm	60
2.36mm	50.7
1.18mm	40.9
600µm	28.8
300µm	13.3
150µm	5.7
75µm	3.7
Material Properties	
Asphalt Cement (PG 58-28)	5.3%
Maximum Relative Density (MRD)	2.496
Air voids	4.0%
Voids in Mineral Aggregate (VMA)	15.5%
Volume Filled with Asphalt (VFA)	74.2%



Ontario Ministry of Transportation

FLEXIBLE PAVEMENT CONDITION EVALUATION FORM

Location From: ______ To: ______

LHRS [ ] BEGINS [.] OFFSET km — Section Length [.] LENGTH km — District [ ]

Survey Date [ ] YEAR [ ] MONTH — PCR [ ] — RCR [.] — Traffic Direction [ ] B : BOTH DIRECTIONS, N : NORTH BOUND, S : SOUTH BOUND, E : EAST BOUND, W : WEST BOUND — Highway [ ]

Contract No. [ ] – [ ] — WP No. [ ] – [ ] – [ ] — Facility [ ] A : ALL LANES, C : COLLECTOR, E : EXPRESS, O : OTHERS (Additional Lanes) — Class [ ] F : FREEWAY, A : ARTERIAL, C : COLLECTOR, L : LOCAL, S : SECONDARY

Ride Condition Rating (at 80 km/h): 10 — 8 EXCELLENT Smooth and pleasant — 6 GOOD Comfortable — 4 FAIR Uncomfortable — 2 POOR Very rough and bumpy — 0 VERY POOR Dangerous at 80 km/h

Pavement		No.	Very Slight 1	Slight 2	Moderate 3	Severe 4	Very Severe 5	Few < 10 1	Intermittent 10-20 2	Frequent 20-50 3	Extensive 50-80 4	Throughout 80-100 5
SURFACE DEFECTS	Ravelling & C. Agg. Loss	1										
	Flushing	2										
SURFACE DEFORMATIONS	Rippling and Shoving	3										
	Wheel Track Rutting	4										
	Distortion	5										
CRACKING – Longitudinal Wheel Track	Single and Multiple	6										
	Alligator	7										
CRACKING – Centre Line	Single and Multiple	8										
	Alligator	9										
CRACKING – Pavement Edge	Single and Multiple	10										
	Alligator	11										
CRACKING – Transverse	Half, Full and Multiple	12										
	Alligator	13										
CRACKING	Longitudinal Meander and Midlane	14										
	Random	15										

(Columns 1–5: SEVERITY OF DISTRESS; columns 6–10: DENSITY OF DISTRESS, Extent of Occurrence,%)

Shoulders – DOMINANT TYPE	✓ ONE	DISTRESS	Severity RIGHT Mod 1	Severity RIGHT Severe 2	Severity LEFT Mod 1	Severity LEFT Severe 2	Density RIGHT 10-30 1	Density RIGHT >30 2	Density LEFT 10-30 1	Density LEFT >30 2
PAVED FULL		Cracking								
		Pavement Edge/ Curb Separation								
PAVED PARTIAL		Distortion								
SURFACE TREATED		Breakup/Separation								
		Edge Break								
PRIMED		Breakup								
GRAVEL										

(SEVERITY OF DISTRESS; DENSITY OF DISTRESS, Extent of Occurrence,%)

Maintenance Treatment		<10 1	10-20 2	20-50 3	50-80 4	>80 5
PAVEMENT	Manual Patching					
	Machine Patching					
	Spray Patching					
	Rout and Seal Cracks					
	Chip Seal					
SHOULDERS	Manual Patching					
	Machine Patching					
	Rout and Seal Cracks					
	Chip Seal					

(EXTENT OF OCCURRENCE, %)

Distress Comments (Items not covered above) ______

Other Comments (e.g. subsections, additional contracts) ______

Evaluated by ______

PH-D-584 86-01

Figure A-1 Pavement Condition Rating Form

105

Figure A-1: Flexible Pavement Condition Evaluation Form

Appendix B: PaLATE Charts for Construction

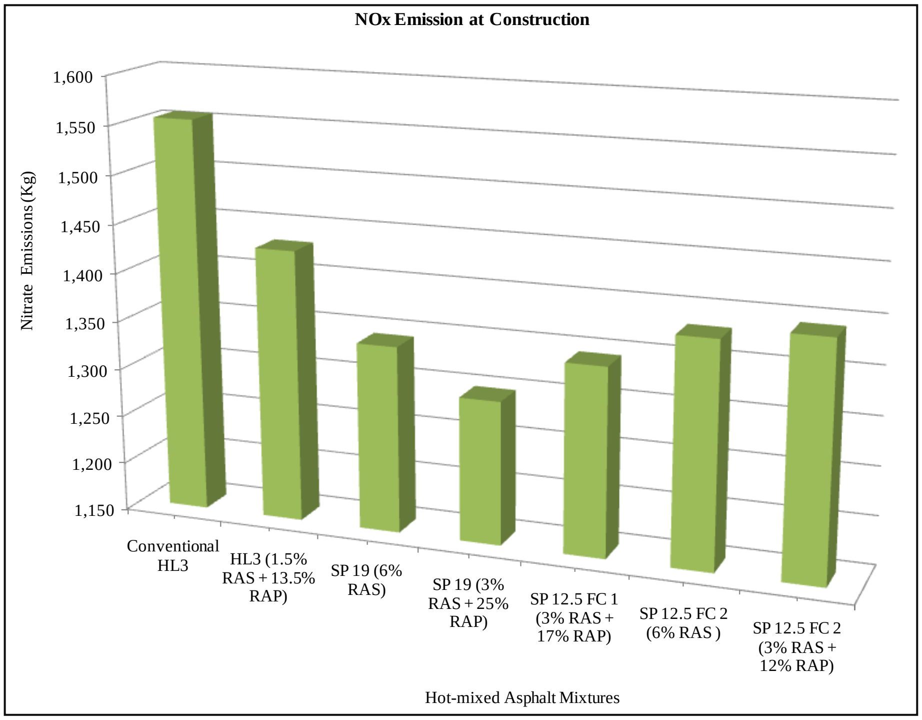


Figure B-2: NO_x Output for Construction

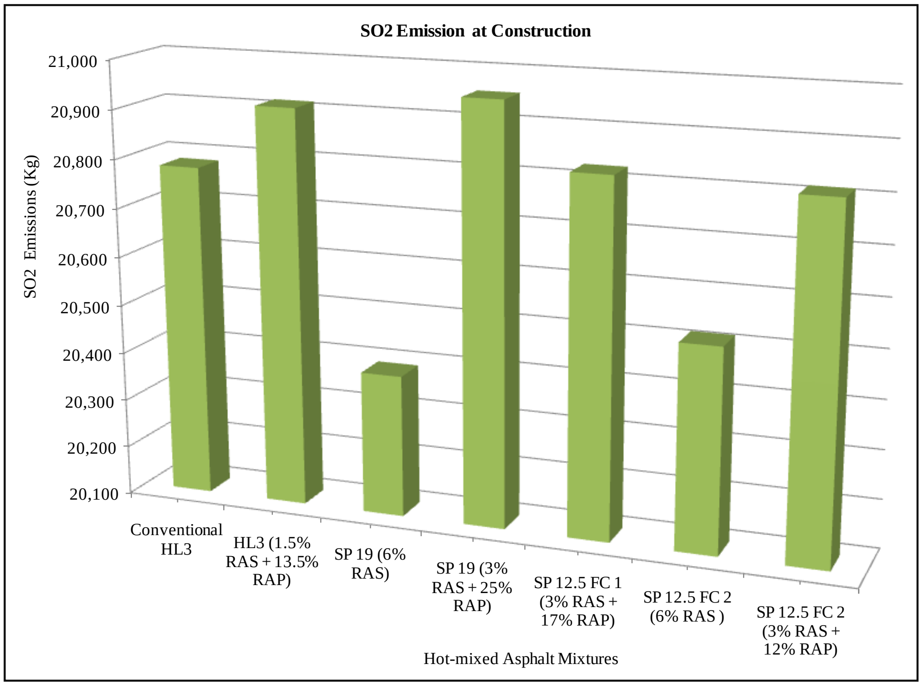


Figure B-3: SO_2 Output for Construction

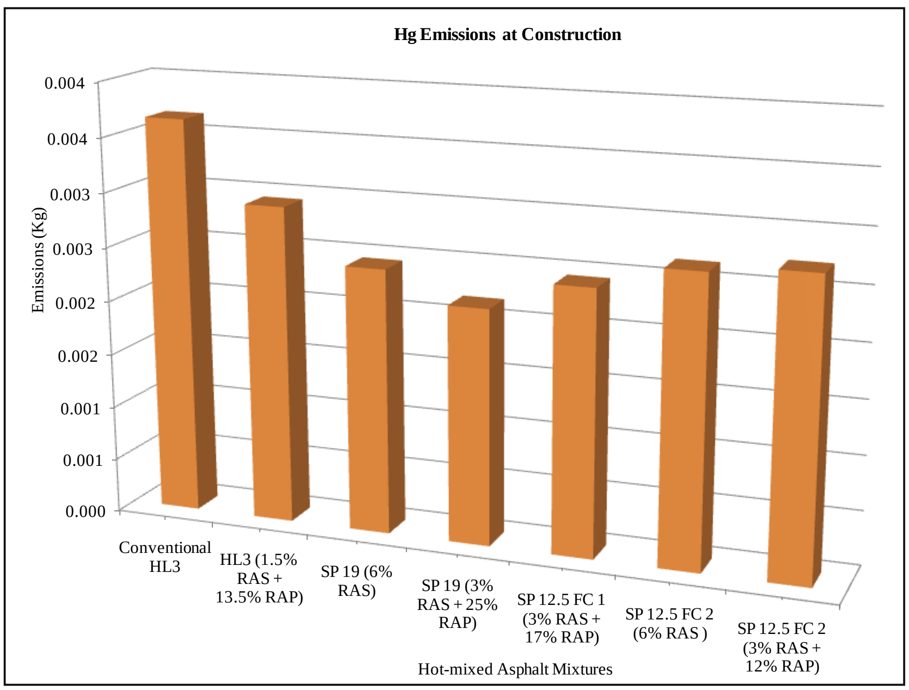


Figure B-4: Hg Output for Construction

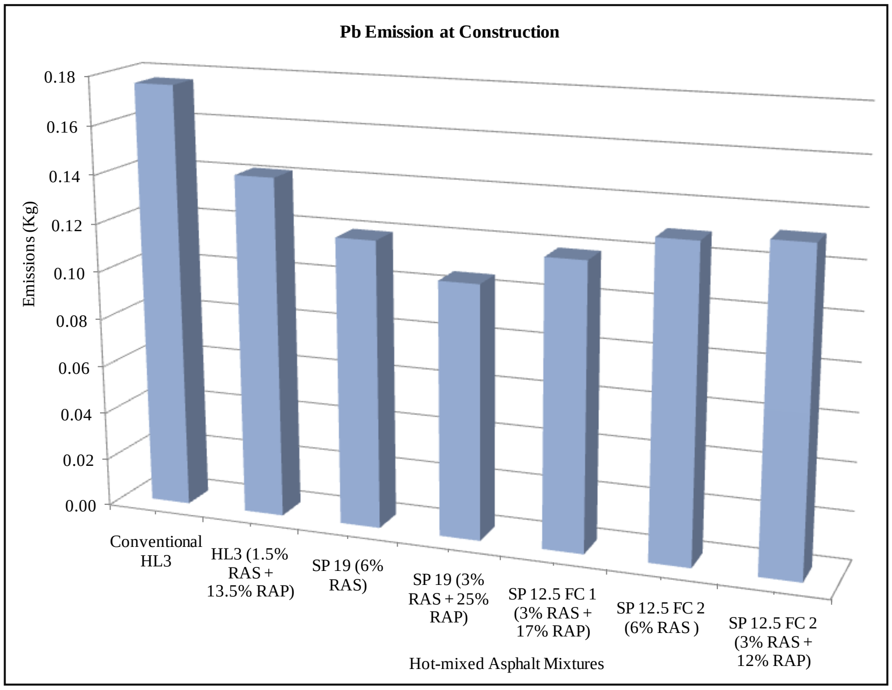


Figure B-5: Pb Output for Construction

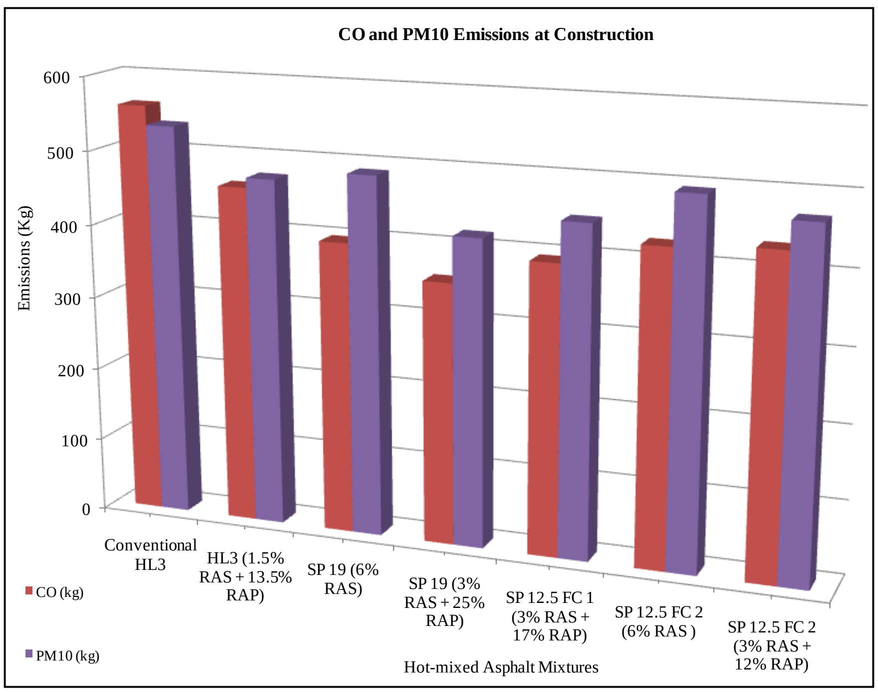


Figure B-6: CO and PM_{10} Output for Construction

Appendix C: Life-Cycle Cost

Table C-3: Life Cycle Cost for Control Mix and Mix 1

Conventional HMA							5%			
Proposed Maintenance Activity	**Year**	**Unit Cost**	**Estimated Quantity Per Km**	**unit cost / Km ($)**	**length/Km**	**Unit Cost**	**PW factor**	**Present Worth Cost**	**Net Present Value (NPV)**	**EUAC**
Conventional HMA	0				1	$ 980,482.32	1.000	$ 980,482.32		
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.864	$ 863.84	$ 981,228.54	$ 7,206,310.44
5% Mill and patch 40 mm	8	$35.00	750	$26,250	1	$ 26,250.00	0.677	$ 17,767.03	$ 992,507.75	$ 3,071,251.97
20% Mill and patch 40 mm	15	$35.00	3000	$105,000	1	$ 105,000.00	0.481	$ 50,506.80	$ 1,004,776.95	$ 1,936,050.20
20% Mill and Patch 40mm	20	$35.00	3000	$105,000	1	$ 105,000.00	0.377	$ 39,573.40	$ 995,397.12	$ 1,597,464.80
Salvage Value	20					-$ 196,096.46	0.377	**-$ 73,906.69**		
								$ 1,015,286.69		

HL3 1.5% RAS, 13.5% RAP							5%			
Proposed Maintenance Activity	**Year**	**Unit Cost**	**Estimated Quantity Per Km**	**unit cost / Km ($)**	**length/Km**	**Unit Cost**	**PW factor**	**Present Worth Cost**	**Net Present Value (NPV)**	**EUAC**
HL3 1.5% RAS 13.5% RAP	0				1	$ 966,662.64	1.000	$ 966,662.64		
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.864	$ 863.84	$ 967,408.86	$ 7,104,816.34
5% Mill and patch 40 mm	5	$35.00	750	$26,250	1	$ 26,250.00	0.784	$ 20,567.56	$ 982,777.86	$ 4,539,938.37
20% Mill and patch 40 mm	9	$35.00	3000	$105,000	1	$ 105,000.00	0.645	$ 67,683.94	$ 1,010,292.31	$ 2,842,762.11
20% Mill and Patch 40mm	15	$35.00	3000	$105,000	1	$ 105,000.00	0.481	$ 50,506.80	$ 990,957.27	$ 1,909,421.81
Mill 90mm/Place 90mm Asphalt Pavement (Overlay)	19	$15.00	3375	$50,625	1	$ 50,625.00	0.396	$ 20,034.03	$ 974,590.79	$ 1,612,850.50
Salvage Value	20					-$ 193,332.53	0.377	**-$ 72,865.00**		
								$ 1,053,453.81		

Table C-4: Life Cycle Cost for Surface Layer Mixes

SP 12.5 FC1 3% RAS, 17% RAP

							5%			
Proposed Maintenance Activity	**Year**	**Unit Cost**	**Estimated Quantity Per Km**	**unit cost / Km ($)**	**length/Km**	**Unit Cost**	**PW factor**	**Present Worth Cost**	**Net Present Value (NPV)**	**EUAC**
SP 12.5 FC1 3% RAS, 17% RAP	0				1	$ 975,583.44	1.000	$ 975,583.44		
Rout and Crack Sealing (200 m/km)	3	$5.00	200.00	$1,000	1	$ 1,000.00	0.864	$ 863.84	$ 976,329.66	$ 7,170,332.23
5% Mill and patch 40 mm	6	$35.00	750.00	$26,250	1	$ 26,250.00	0.746	$ 19,588.15	$ 990,200.42	$ 3,901,735.60
20% Mill and Patch 40mm	10	$35.00	3000.00	$105,000	1	$ 105,000.00	0.614	$ 64,460.89	$ 1,015,156.84	$ 2,629,349.09
20% Mill and Patch 40mm	15	$35.00	3000.00	$105,000	1	$ 105,000.00	0.481	$ 50,506.80	$ 999,878.07	$ 1,926,610.82
SuperPave 12.5mm,FC1 40mm	20	$55.37	1512.00	$83,719	1	$ 83,719.44	0.377	$ 31,552.98	$ 987,475.42	$ 1,584,751.66
Salvage Value	20					-$ 195,116.69	0.377	**-$ 73,537.43**		
								$ 1,069,018.67		

SP 12.5 FC2 6% RAS

							5%			
Proposed Maintenance Activity	**Year**	**Unit Cost**	**Estimated Quantity Per Km**	**unit cost / Km ($)**	**length/Km**	**Unit Cost**	**PW factor**	**Present Worth Cost**	**Net Present Value (NPV)**	**EUAC**
SP 12.5 FC2 6% RAS	0				1	$ 1,005,324.48	1.000	$ 1,005,324.48		
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.864	$ 863.84	$ 1,006,070.70	$ 7,388,755.52
5% Mill and patch 40 mm	6	$35.00	750	$26,250	1	$ 26,250.00	0.746	$ 19,588.15	$ 1,019,941.46	$ 4,018,925.69
20% Mill and Patch 40mm	9	$35.00	3000	$105,000	1	$ 105,000.00	0.645	$ 67,683.94	$ 1,048,954.15	$ 2,951,548.86
20% Mill and Patch 40mm	15	$35.00	3000	$105,000	1	$ 105,000.00	0.481	$ 50,506.80	$ 1,029,619.11	$ 1,983,917.21
Salvage Value	20					-$ 201,064.90	0.377	**-$ 75,779.24**		
								$ 1,068,187.96		

SP 12.5 FC2 3% RAS, 12% RAP							5%			
Proposed Maintenance Activity	**Year**	**Unit Cost**	**Estimated Quantity Per Km**	**unit cost / Km ($)**	**length/Km**	**Unit Cost**	**PW factor**	**Present Worth Cost**	**Net Present Value (NPV)**	**EUAC**
SP 12.5 FC2 3% RAS, 12% RAP	0				1	$ 991,232.64	1.000	$ 991,232.64		
Rout and Crack Sealing (200 m/km)	3	$5.00	200.00	$1,000	1	$ 1,000.00	0.864	$ 863.84	$ 991,978.86	$ 7,285,262.63
5% Mill and patch 40 mm	6	$35.00	750.00	$26,250	1	$ 26,250.00	0.746	$ 19,588.15	$ 1,005,849.62	$ 3,963,398.92
20% Mill and Patch 40mm	10	$35.00	3000.00	$105,000	1	$ 105,000.00	0.614	$ 64,460.89	$ 1,030,806.04	$ 2,669,881.95
20% Mill and Patch 40mm	15	$35.00	3000.00	$105,000	1	$ 105,000.00	0.481	$ 50,506.80	$ 1,015,527.27	$ 1,956,764.41
SuperPave 12.5mm,FC2 40mm	19	$64.72	1512.00	$97,857	1	$ 97,856.64	0.396	$ 38,725.20	$ 1,006,557.51	$ 1,665,752.24
Salvage Value	20					-$ 198,246.53	0.377	**-$ 74,717.03**		
								$ 1,090,660.48		

Table C-5: Life Cycle Cost for Binder Layer Mixes

SP 19 6% RAS

Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	unit cost / Km ($)	length/Km	Unit Cost	5%			
							PW factor	Present Worth Cost	Net Present Value (NPV)	EUAC
SP 19 6% RAS	0				1	$ 919,214.10	1.000	$ 919,214.10		
Rout and Crack Sealing (200 m/km)	4	$5.00	200	$1,000	1	$ 1,000.00	0.823	$ 822.70	$ 919,890.94	$ 5,188,402.59
5% Mill and patch 40 mm	9	$35.00	750	$26,250	1	$ 26,250.00	0.645	$ 16,920.98	$ 930,121.52	$ 2,617,177.41
20% Mill and Patch 40mm	15	$35.00	3000	$105,000	1	$ 105,000.00	0.481	$ 50,506.80	$ 943,508.73	$ 1,817,995.79
20% Mill and Patch 40mm	19	$35.00	3000	$105,000	1	$ 105,000.00	0.396	$ 41,552.07	$ 935,657.66	$ 1,548,420.06
Salvage Value	20					-$ 183,842.82	0.377	**-$ 69,288.43**		
								$ 959,728.22		

SP 19 3% RAS, 25% RAP

Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	unit cost / Km ($)	length/Km	Unit Cost	5%			
							PW factor	Present Worth Cost	Net Present Value (NPV)	EUAC
SP 19 3% RAS, 25% RAP	0				1	$ 863,768.16	1.000	$ 863,768.16		
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.864	$ 863.84	$ 864,514.38	$ 6,349,141.66
5% Mill and patch 40 mm	6	$35.00	750	$26,250	1	$ 26,250.00	0.746	$ 19,588.15	$ 878,385.14	$ 3,461,144.33
20% Mill and Patch 40mm	9	$35.00	3000	$105,000	1	$ 105,000.00	0.645	$ 67,683.94	$ 907,397.83	$ 2,553,237.46
20% Mill and Patch 40mm	15	$35.00	3000	$105,000	1	$ 105,000.00	0.481	$ 50,506.80	$ 888,062.79	$ 1,711,160.02
Salvage Value	20					-$ 172,753.63	0.377	**-$ 65,109.03**		
								$ 937,301.86		

Appendix D: HMA Mix Designs

Table 1 Gradation of the HL3 RAS mix

JOB MIX FORMULA—GRADATION PERCENT PASSING														
% AC/Sieve size (mm)	%AC	26.5	19.0	16.0	13.2	9.5	6.7	4.75	2.36	1.18	0.60	0.30	0.15	0.075
JMF	5*	100	100	100	99	82.7	64.8	55	43.7	30.3	20.2	11.5	6.8	4.6

Table 2 Design Features of HL3 RAS

Marshall Test Result		Requirements	Selected
Percentage Air Voids		4.0+/-0.5	4.0
Flow (min)[0.25mm]@ 3.5% Air Voids		8	10.5
Stability(min) N		8900	16750
Percentage Voids in Mineral Aggregates		15.0	15
Aggregate Types	**Percentage**	**Mix Properties**	**Percentage**
Coarse Aggregate #1	40.3	Asphalt Cement (A.C) in RAP	6.87
Coarse Aggregate #2	-	RAP PEN	N/A
Coarse Aggregate #3	-	Bulk Relative Density, BRD	2.412
Fine Aggregate # 1	8.0	Maximum Relative Density ,MRD	2.513
Fine Aggregate # 2	36.7	Specific Gravity, Gb	2.696
RAP	15.0**		
Aggregate Types		**Source**	
Coarse Aggregate # 1		Hiedelberg (HL3 stone)	
Fine Aggregate # 1		Hiedelberg (Screening)	
Fine Aggregate # 2		Hiedelberg (Asphalt Sand)	
RAP # 1		Hiedelberg (16 mm RAP)	
Shingles		Miller Paving Ltd	
Asphalt Cement		McAsphalt (PG 58-28)	

*AC from RAP=1.305; New AC=3.9695%

**% RAP indicated contains 13.5% RAP and 1.5% RAS

Figure D-7: Mix 1: HL 3 1.5% RAS and 13.5% RAP

HOT MIX ASPHALT DESIGN BLEND SHEET

Mix: SP19.0 / Base Course 'Superpave Method | Supplier Research | RAP AC 4.80 | 27.00

Project	Various	Product No				Traffic Categroy	
Project No	M2010	% RAP	6.0	Stability			
Client	Research	% AC-RAP	27.00	Flow		E	
Mix No	1019B	DP (Spec)	0.6-1.2	VMA	13	Estmd for AV=4	
Mix Type	SP19.0 / Base Course	Nini (Spec)	9	VFA	65-75	Gmb	2.455
Date	25-Jan-10	Ndes (Spec)	125	TSR	>=80%	Gmm	2.556
% AC	4.9	Nmax (Spec)	205	VCA_{max}		VMA	12.93
AC Grade	PG 52-40 - Conventional	% Fines	3.0	VCA_{dry}		DP	1.72
Supplier	McAsphalt	FF Ratio	2.93	Voids %	4.0+/-0.5	Pbe	3.747
Comp. G_{sb}	2.681	Anti-Stripp.		Min AC	5.0	Gse	2.769

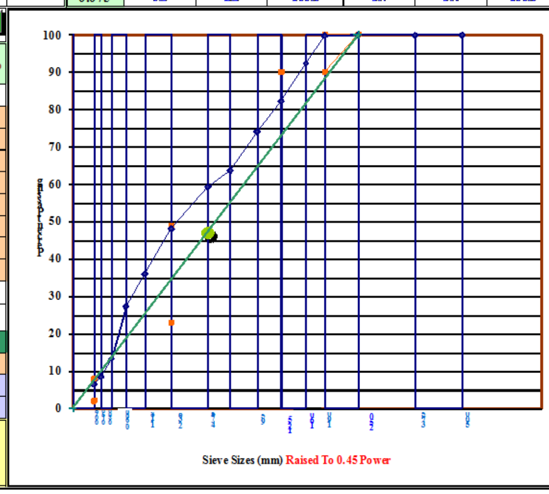
	MATERIAL	SOURCE	INV. #	% AGG	% VOL	Gsb	Gsa	Abs
CA1	HL8 Stone	Carden	O11-123	39.5	39.1	2.658	2.718	0.830
CA2	1/4" Chips	Carden	O11-123	13.8	13.7	2.670	2.706	0.500
CA3				0.0	0.0	1.000	1.000	0.000
FA1	Mfg. Sand	Dufferin	B12-127	28.8	29.9	2.788	2.835	0.592
FA2	IKO Sand	IKO	C01-148	11.9	12.2	2.750	2.788	0.500
FA3				0.0	0.0	1.000	1.000	0.000
RAP1	1/2" RAP	Markham		0.0	0.0	2.708	2.708	0.000
RAP2	Shingles	Markham		6.0	5.1	2.300	2.300	0.000
Film Thick	#REF!		Total:	100.0	100.0	Gmm	2.509	

	COARSE AGGREGATES			FINE AGGREGATES			RAP	
	CA1	CA2	CA3	FA1	FA2	FA3	RAP1	RAP2
Sieve Sizes (mm)	HL8 Stone Carden	1/4" Chips Carden		Mfg. Sand Dufferin	IKO Sand IKO		1/2" RAP Markham	Shingles Markham
Lab No.								
50.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
37.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	95.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0
16.0	76.9	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	51.8	100.0	100.0	100.0	100.0	100.0	100.0	99.5
9.5	31.4	100.0	100.0	100.0	100.0	100.0	96.8	99.2
6.7	10.1	87.7	100.0	100.0	99.9	100.0	84.0	98.0
4.75	1.6	82.0	100.0	99.8	99.8	100.0	70.0	96.2
2.36	0.7	24.6	100.0	85.6	99.5	100.0	50.1	92.0
1.18	0.7	9.6	100.0	54.3	98.9	100.0	35.2	75.3
0.600	0.6	4.7	100.0	31.8	96.1	100.0	25.6	52.7
0.300	0.6	3.2	100.0	17.4	18.5	100.0	18.1	41.7
0.150	0.6	2.6	100.0	8.6	5.9	100.0	12.8	29.2
0.075	0.5	2.3	100.0	3.7	3.4	100.0	9.2	22.6

Please Select a Specification | Superpave | SP19.0 | Re-Calculate

Sieve Sizes (mm)	BLEND	LOW	HIGH	FINAL BLEND	Aggregate	%	Agg. Proportion
50.0	100.0			100.0	% Total CA		
37.5	100.0			100.0	% Total FA		
25.0	100.0	100.0	100.0	100.0	CA1	74.5	39.5
19.0	98.2	90.0	100.0	99.8	CA2	25.5	13.8
16.0	90.9			92.4	CA3		0.0
12.5	80.9		90.0	82.3	FA1	71.4	28.8
9.5	72.9			74.1	FA2	28.6	11.9
6.7	62.7			63.7	FA3		0.0
4.75	58.3			59.3	Total FA	100.0	
2.36	45.7	23.0	49.0	48.0	RAP1	4.80	0.0
1.18	33.5			36.0	RAP2	27.00	6.0
0.600	24.6			27.3	Aggs. Total		100.0
0.300	10.4			13.2	Total CA	100.0	
0.150	5.5			8.4	Total AC		4.90
0.075	3.3	2.0	8.0	6.4	New AC		3.28

Notes:



Bailey's Method	
Blend Characteristics	
NMAS	19.0mm
PCS	4.75mm
SCS	1.18mm
TCS	0.300mm
Half Sieve	9.5mm
CA Ratio	0.52
Range	0.60-0.75
FAc Ratio	0.55
Range	0.35-0.50
FAf Ratio	0.42
Range	0.35-0.50
LUW_{CA}	1.0
RUW_{CA}	1.0
LUW_{FA}	1.0
RUW_{FA}	1.0
Programmer	Narayan

Figure D-8: Mix 2: SP19 6% RAS

HOT MIX ASPHALT DESIGN BLEND SHEET

Mix: SP19.0 / Base Course 'Superpave Method — Supplier: Research — RAP AC: 4.80 | 27.00

Project	Various	Product No				Traffic Categroy	
Project No	M2010	% RAP	28.0	Stability			
Client	Research	% AC-RAP	7.18	Flow		E	
Mix No	1009B	DP (Spec)	0.6-1.2	VMA	13	Estmd for AV=4	
Mix Type	SP19.0 / Base Course	Nini (Spec)	9	VFA	65-75	Gmb	2.455
Date	20-Jan-10	Ndes (Spec)	125	TSR	>=80%	Gmm	2.556
% AC	4.9	Nmax (Spec)	205	VCA_{max}		VMA	13.29
AC Grade	PG 52-34 - Conventional	% Fines	0.5	VCA_{dry}		DP	1.32
Supplier	McAsphalt	FF Ratio	3.12	Voids %	4.0+/-0.5	Pbe	3.898
Comp. Gsb	2.692	Anti-Stripp.		Min AC	5.0	Gse	2.769

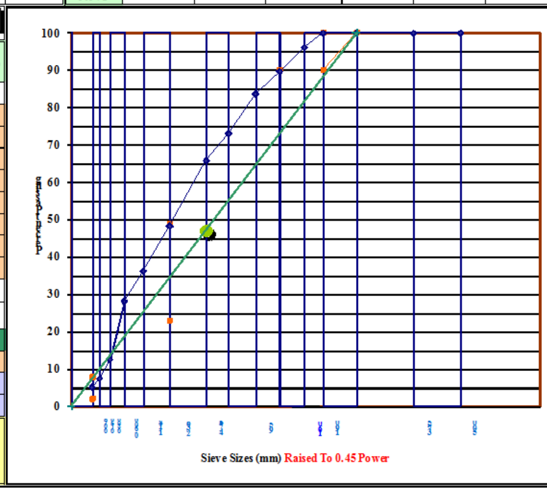
	MATERIAL	SOURCE	INV. #	% AGG	% VOL	Gsb	Gsa	Abs
CA1	HL8 Stone	Carden	O11-123	25.1	24.8	2.658	2.718	0.830
CA2	1/4" Chips	Carden	O11-123	17.1	16.9	2.670	2.706	0.500
CA3				0.0	0.0	1.000	1.000	0.000
FA1	Mfg. Sand	Dufferin	B12-127	16.1	16.7	2.788	2.835	0.592
FA2	IKO Sand	IKO	C01-148	13.7	14.0	2.750	2.788	0.500
FA3				0.0	0.0	1.000	1.000	0.000
RAP1	1/2" RAP	Markham		25.0	25.1	2.708	2.708	0.000
RAP2	Shingles	Markham		3.0	2.6	2.300	2.300	0.000
Film Thick	#REF!		Total:	100.0	100.0	Gmm	2.518	

Sieve Sizes (mm)	CA1	CA2	CA3	FA1	FA2	FA3	RAP1	RAP2
	COARSE AGGREGATES			FINE AGGREGATES			RAP	
	HL8 Stone Carden	1/4" Chips Carden		Mfg. Sand Dufferin	IKO Sand IKO		1/2" RAP Markham	Shingles Markham
Lab No.								
50.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
37.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	95.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0
16.0	76.9	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	51.8	100.0	100.0	100.0	100.0	100.0	100.0	99.5
9.5	31.4	100.0	100.0	100.0	100.0	100.0	96.8	99.2
6.7	10.1	90.0	100.0	100.0	100.0	100.0	84.0	98.0
4.75	1.6	82.0	100.0	99.8	99.8	100.0	70.0	96.2
2.36	0.7	24.6	100.0	85.6	99.5	100.0	50.1	92.0
1.18	0.7	9.6	100.0	54.3	98.9	100.0	35.2	75.3
0.600	0.6	4.7	100.0	31.8	96.1	100.0	25.6	52.7
0.300	0.6	3.2	100.0	17.4	18.5	100.0	18.1	41.7
0.150	0.6	2.6	100.0	8.6	5.9	100.0	12.8	29.2
0.075	0.5	2.3	100.0	3.7	3.4	100.0	9.2	22.6

Please Select a Specification — Superpave — SP19.0 — Re-Calculate

Sieve Sizes (mm)	BLEND	LOW	HIGH	FINAL BLEND	Aggregate	%	Agg. Proportion
50.0	100.0			100.0	% Total CA		
37.5	100.0			100.0	% Total FA		
25.0	100.0	100.0	100.0	100.0	CA1		25.1
19.0	98.9	90.0	100.0	100.0	CA2		17.1
16.0	94.2			96.1	CA3		0.0
12.5	87.9		90.0	89.7	FA1		16.1
9.5	82.0			83.6	FA2		13.7
6.7	71.7			73.1	FA3		0.0
4.75	64.6			65.9	Total FA	0.0	
2.36	47.1	23.0	49.0	48.3	RAP1	4.80	25.0
1.18	35.2			36.2	RAP2	27.00	3.0
0.600	27.2			28.1	Aggs. Total		100.0
0.300	11.8			12.5	Total CA	0.0	
0.150	6.9			7.5	Total AC		4.90
0.075	4.6	2.0	8.0	5.2	New AC		2.89

Notes:



Bailey's Method	
Blend Characteristics	
NMAS	19.0mm
PCS	4.75mm
SCS	1.18mm
TCS	0.300mm
Half Sieve	9.5mm
CA Ratio	0.52
Range	0.60-0.75
FAc Ratio	0.55
Range	0.35-0.50
FAf Ratio	0.42
Range	0.35-0.50
LUW_{CA}	1.0
RUW_{CA}	1.0
LUW_{FA}	1.0
RUW_{FA}	1.0
Programmer	Narayan

Figure D-9: Mix 3: SP19 3% RAS and 25% RAP

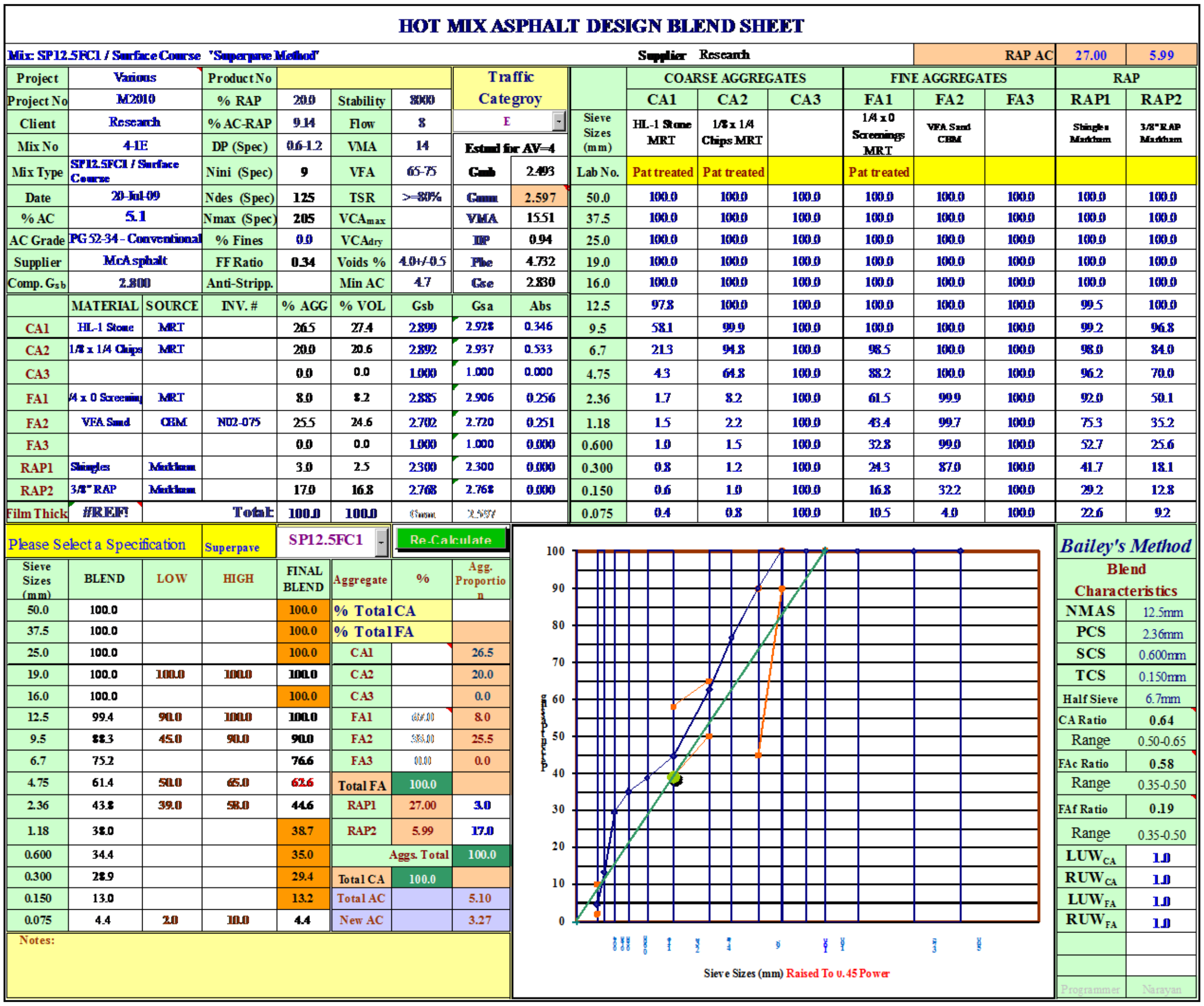


Figure D-10: Mix 4: SP12.5 FC1 3% RAS and 17% RAP

HOT MIX ASPHALT DESIGN BLEND SHEET

Mix: SP12.5FC2 / Surface Course 'Superpave Method'   Supplier Research   RAP AC 27.00 5.99

Project	Various	Product No				Traffic Categroy	
Project No	M2010	% RAP	6.0	Stability	8900		
Client	Research	% AC-RAP	27.00	Flow	8	E	
Mix No	18-1A	DP (Spec)	0.6-1.2	VMA	14	Estmd for AV=4	
Mix Type	SP12.5FC2 / Surface Course	Nini (Spec)	9	VFA	65-75	Gmb	2.536
Date	27-Jan-10	Ndes (Spec)	125	TSR	>=80%	Gmm	2.641
% AC	5.2	Nmax (Spec)	205	VCA_{max}		VMA	15.99
AC Grade	PG 52-40 - Conventional	% Fines	0.0	VCA_{dry}		DP	0.82
Supplier	McAsphalt	FF Ratio	1.13	Voids %	4.0+/-0.5	Pbe	4.848
Comp. Gsb	2.861	Anti-Stripp.		Min AC	5.0	Gse	2.891

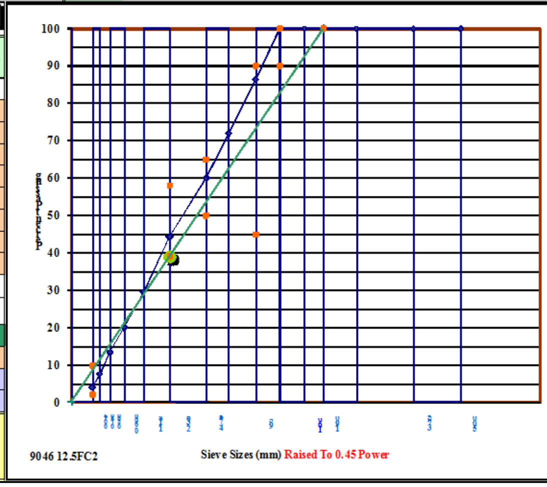
	MATERIAL	SOURCE	INV. #	% AGG	% VOL	Gsb	Gsa	Abs
CA1	HL1 Stone	MRT		35.6	36.0	2.899	2.928	0.346
CA2	1/8 x 1/4 Chips	MRT		14.0	14.1	2.892	2.937	0.533
CA3				0.0	0.0	1.000	1.000	0.000
FA1	/4 x 0 Screening	MRT		15.0	15.1	2.885	2.906	0.256
FA2	Mfg. Sand	MRT		29.4	30.1	2.934	2.969	0.402
FA3				0.0	0.0	1.000	1.000	0.000
RAP1	Shingles	Markham		6.0	4.8	2.300	2.300	0.000
RAP2	3/8" RAP	Markham		0.0	0.0	2.790	2.790	0.000
Film Thick	#REF!		Total:	100.0	100.0	Gmm	2.641	

Sieve Sizes (mm)	COARSE AGGREGATES CA1	CA2	CA3	FINE AGGREGATES FA1	FA2	FA3	RAP RAP1	RAP2
	HL1 Stone MRT	1/8 x 1/4 Chips MRT		1/4 x 0 Screenings MRT	Mfg. Sand MRT		Shingles Markham	3/8" RAP Markham
Lab No.	Pat treated	Pat treated		Pat treated	Pat treated			
50.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
37.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
16.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	97.8	100.0	100.0	100.0	100.0	100.0	99.5	100.0
9.5	58.1	99.9	100.0	100.0	100.0	100.0	99.2	96.8
6.7	21.3	94.8	100.0	98.5	100.0	100.0	98.0	84.0
4.75	4.3	64.8	100.0	88.2	100.0	100.0	96.2	70.0
2.36	1.7	8.2	100.0	61.5	92.0	100.0	92.0	50.1
1.18	1.5	2.2	100.0	43.4	58.4	100.0	75.3	35.2
0.600	1.0	1.5	100.0	32.8	37.7	100.0	52.7	25.6
0.300	0.8	1.2	100.0	24.3	22.3	100.0	41.7	18.1
0.150	0.6	1.0	100.0	16.8	9.3	100.0	29.2	12.8
0.075	0.4	0.8	100.0	10.5	2.4	100.0	22.6	9.2

Please Select a Specification   Superpave   SP12.5FC2   Re-Calculate

Sieve Sizes (mm)	BLEND	LOW	HIGH	FINAL BLEND	Aggregate	%	Agg. Proportion
50.0	100.0			100.0	% Total CA		
37.5	100.0			100.0	% Total FA		
25.0	100.0			100.0	CA1		35.6
19.0	100.0	100.0	100.0	100.0	CA2		14.0
16.0	100.0			100.0	CA3		0.0
12.5	99.2	90.0	100.0	100.0	FA1	[illegible]	15.0
9.5	85.0	45.0	90.0	86.4	FA2	[illegible]	29.4
6.7	70.9			72.1	FA3	00.00	0.0
4.75	59.0	50.0	65.0	60.0	Total FA	100.0	
2.36	43.5	39.0	58.0	44.3	RAP1	27.00	6.0
1.18	29.0			29.5	RAP2	5.99	0.0
0.600	19.7			20.1	Aggs. Total		100.0
0.300	13.2			13.4	Total CA	100.0	
0.150	7.4			7.5	Total AC		5.20
0.075	3.9	2.0	10.0	4.0	New AC		3.58

Notes:



Bailey's Method	
Blend Characteristics	
NMAS	12.5mm
PCS	2.36mm
SCS	0.600mm
TCS	0.150mm
Half Sieve	6.7mm
CA Ratio	2.80
Range	0.50-0.65
FAc Ratio	0.65
Range	0.35-0.50
FAf Ratio	0.48
Range	0.35-0.50
LUW_{CA}	1.0
RUW_{CA}	1.0
LUW_{FA}	1.0
RUW_{FA}	1.0
Programmer	Narayan

Figure D-11: Mix 5: SP12.5 FC2 6% RAS

HOT MIX ASPHALT DESIGN BLEND SHEET

Mix: SP12.5FC2 / Surface Course 'Superpave Method'   Supplier Research   RAP AC 27.00 5.99

Project	Various	Product No				Traffic Categroy	
Project No	M2010	% RAP	15.0	Stability	8900		
Client	Research	%AC-RAP	10.19	Flow	8	E	
Mix No	3-1B	DP (Spec)	0.6-1.2	VMA	14	Estmd for AV=4	
Mix Type	SP12.5FC2 / Surface	Nini (Spec)	9	VFA	65-75	Gmb	2.539
Date	27-Jan-10	Ndes (Spec)	125	TSR	>=80%	Gmm	2.645
% AC	5.2	Nmax (Spec)	205	VCAmax		VMA	16.01
AC Grade	PG 52-34 - Conventional	% Fines	0.0	VCAdry		DP	0.86
Supplier	McAsphalt	FF Ratio	1.18	Voids %	4.0+/-0.5	Pbe	4.849
Comp. Gsb	2.866	Anti-Stripp.		Min AC	5.0	Gse	2.896

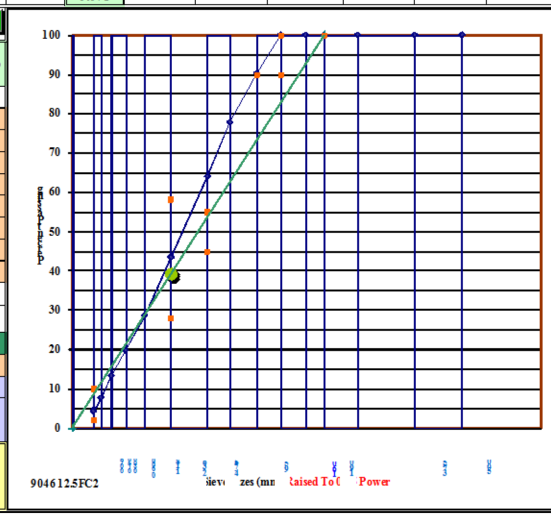
	MATERIAL	SOURCE	INV. #	% AGG	% VOL	Gsb	Gsa	Abs
CA1	HL1 Stone	MRT		25.7	26.0	2.899	2.928	0.346
CA2	1/8 x 1/4 Chips	MRT		20.0	20.1	2.892	2.937	0.533
CA3				0.0	0.0	1.000	1.000	0.000
FA1	/4 x 0 Screening	MRT		14.0	14.1	2.885	2.906	0.256
FA2	Mfg. Sand	MRT		25.3	25.9	2.934	2.969	0.402
FA3				0.0	0.0	1.000	1.000	0.000
RAP1	Shingles	Carden		3.0	2.4	2.300	2.300	0.000
RAP2	3/8" RAP	Markham		12.0	11.6	2.768	2.768	0.000
Film Thick	#REF!		Total:	100.0	100.0	Gmm	2.645	

	COARSE AGGREGATES			FINE AGGREGATES			RAP	
	CA1	CA2	CA3	FA1	FA2	FA3	RAP1	RAP2
Sieve Sizes (mm)	HL1 Stone MRT	1/8 x 1/4 Chips MRT		1/4 x 0 Screenings MRT	Mfg. Sand MRT		Shingles Carden	3/8" RAP Markham
Lab No.	Pat treated	Pat treated		Pat treated	Pat treated			
50.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
37.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
25.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
16.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
12.5	97.8	100.0	100.0	100.0	100.0	100.0	99.5	100.0
9.5	58.1	99.9	100.0	100.0	100.0	100.0	99.2	96.8
6.7	21.3	94.8	100.0	98.5	100.0	100.0	98.0	84.0
4.75	4.3	64.8	100.0	88.2	100.0	100.0	96.2	70.0
2.36	1.7	8.2	100.0	61.5	92.0	100.0	92.0	50.1
1.18	1.5	2.2	100.0	43.4	58.4	100.0	75.3	35.2
0.600	1.0	1.5	100.0	32.8	37.7	100.0	52.7	25.6
0.300	0.8	1.2	100.0	24.3	22.3	100.0	41.7	18.1
0.150	0.6	1.0	100.0	16.8	9.3	100.0	29.2	12.8
0.075	0.4	0.8	100.0	10.5	2.4	100.0	22.6	9.2

Please Select a Specification | Superpave | SP12.5FC2 | Re-Calculate

Sieve Sizes (mm)	BLEND	LOW	HIGH	FINAL BLEND	Aggregate	%	Agg. Proportion
50.0	100.0			100.0	% Total CA		
37.5	100.0			100.0	% Total FA		
25.0	100.0			100.0	CA1		25.7
19.0	100.0	100.0	100.0	100.0	CA2		20.0
16.0	100.0			100.0	CA3		0.0
12.5	99.4	90.0	100.0	100.0	FA1	52.00	14.0
9.5	88.8		90.0	90.2	FA2	45.00	25.3
6.7	76.5			77.7	FA3	0.00	0.0
4.75	63.0	45.0	55.0	64.0	Total FA	100.0	
2.36	42.7	28.0	58.0	43.4	RAP1	27.00	3.0
1.18	28.2			28.6	RAP2	5.99	12.0
0.600	19.3			19.6		Aggs. Total	100.0
0.300	12.9			13.1	Total CA	100.0	
0.150	7.5			7.6	Total AC		5.20
0.075	4.1	2.0	10.0	4.2	New AC		3.67

Notes:



Bailey's Method	
Blend Characteristics	
NMAS	12.5mm
PCS	2.36mm
SCS	0.600mm
TCS	0.150mm
Half Sieve	6.7mm
CA Ratio	2.80
Range	0.50-0.65
FAc Ratio	0.65
Range	0.35-0.50
FAf Ratio	0.48
Range	0.35-0.50
LUW_{CA}	1.0
RUW_{CA}	1.0
LUW_{FA}	1.0
RUW_{FA}	1.0
Programmer	Narayan

Figure D-12: Mix 6: SP12.5 FC2 3% RAS and 12% RAP

Appendix E: Statistical Analysis Tables

F-Test Table

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IV. Percentage Points of the F Distribution (*continued*)

$F_{0.05,\nu_1,\nu_2}$

ν_2 \ ν_1	1	2	3	4	5	6	7	8	9	10	12	15	20	24	30	40	60	120	∞
	161.4	199.5	215.7	224.6	230.2	234.0	236.8	238.9	240.5	241.9	243.9	245.9	248.0	249.1	250.1	251.1	252.2	253.3	254.3
2	18.51	19.00	19.16	19.25	19.30	19.33	19.35	19.37	19.38	19.40	19.41	19.43	19.45	19.45	19.46	19.47	19.48	19.49	19.50
3	10.13	9.55	9.28	9.12	9.01	8.94	8.89	8.85	8.81	8.79	8.74	8.70	8.66	8.64	8.62	8.59	8.57	8.55	8.53
4	7.71	6.94	6.59	6.39	6.26	6.16	6.09	6.04	6.00	5.96	5.91	5.86	5.80	5.77	5.75	5.72	5.69	5.66	5.63
5	6.61	5.79	5.41	5.19	5.05	4.95	4.88	4.82	4.77	4.74	4.68	4.62	4.56	4.53	4.50	4.46	4.43	4.40	4.36
6	5.99	5.14	4.76	4.53	4.39	4.28	4.21	4.15	4.10	4.06	4.00	3.94	3.87	3.84	3.81	3.77	3.74	3.70	3.67
7	5.59	4.74	4.35	4.12	3.97	3.87	3.79	3.73	3.68	3.64	3.57	3.51	3.44	3.41	3.38	3.34	3.30	3.27	3.23
8	5.32	4.46	4.07	3.84	3.69	3.58	3.50	3.44	3.39	3.35	3.28	3.22	3.15	3.12	3.08	3.04	3.01	2.97	2.93
9	5.12	4.26	3.86	3.63	3.48	3.37	3.29	3.23	3.18	3.14	3.07	3.01	2.94	2.90	2.86	2.83	2.79	2.75	2.71
10	4.96	4.10	3.71	3.48	3.33	3.22	3.14	3.07	3.02	2.98	2.91	2.85	2.77	2.74	2.70	2.66	2.62	2.58	2.54
11	4.84	3.98	3.59	3.36	3.20	3.09	3.01	2.95	2.90	2.85	2.79	2.72	2.65	2.61	2.57	2.53	2.49	2.45	2.40
12	4.75	3.89	3.49	3.26	3.11	3.00	2.91	2.85	2.80	2.75	2.69	2.62	2.54	2.51	2.47	2.43	2.38	2.34	2.30
13	4.67	3.81	3.41	3.18	3.03	2.92	2.83	2.77	2.71	2.67	2.60	2.53	2.46	2.42	2.38	2.34	2.30	2.25	2.21
14	4.60	3.74	3.34	3.11	2.96	2.85	2.76	2.70	2.65	2.60	2.53	2.46	2.39	2.35	2.31	2.27	2.22	2.18	2.13
15	4.54	3.68	3.29	3.06	2.90	2.79	2.71	2.64	2.59	2.54	2.48	2.40	2.33	2.29	2.25	2.20	2.16	2.11	2.07
16	4.49	3.63	3.24	3.01	2.85	2.74	2.66	2.59	2.54	2.49	2.42	2.35	2.28	2.24	2.19	2.15	2.11	2.06	2.01
17	4.45	3.59	3.20	2.96	2.81	2.70	2.61	2.55	2.49	2.45	2.38	2.31	2.23	2.19	2.15	2.10	2.06	2.01	1.96
18	4.41	3.55	3.16	2.93	2.77	2.66	2.58	2.51	2.46	2.41	2.34	2.27	2.19	2.15	2.11	2.06	2.02	1.97	1.92
19	4.38	3.52	3.13	2.90	2.74	2.63	2.54	2.48	2.42	2.38	2.31	2.23	2.16	2.11	2.07	2.03	1.98	1.93	1.88
20	4.35	3.49	3.10	2.87	2.71	2.60	2.51	2.45	2.39	2.35	2.28	2.20	2.12	2.08	2.04	1.99	1.95	1.90	1.84
21	4.32	3.47	3.07	2.84	2.68	2.57	2.49	2.42	2.37	2.32	2.25	2.18	2.10	2.05	2.01	1.96	1.92	1.87	1.81
22	4.30	3.44	3.05	2.82	2.66	2.55	2.46	2.40	2.34	2.30	2.23	2.15	2.07	2.03	1.98	1.94	1.89	1.84	1.78
23	4.28	3.42	3.03	2.80	2.64	2.53	2.44	2.37	2.32	2.27	2.20	2.13	2.05	2.01	1.96	1.91	1.86	1.81	1.76
24	4.26	3.40	3.01	2.78	2.62	2.51	2.42	2.36	2.30	2.25	2.18	2.11	2.03	1.98	1.94	1.89	1.84	1.79	1.73
25	4.24	3.39	2.99	2.76	2.60	2.49	2.40	2.34	2.28	2.24	2.16	2.09	2.01	1.96	1.92	1.87	1.82	1.77	1.71
26	4.23	3.37	2.98	2.74	2.59	2.47	2.39	2.32	2.27	2.22	2.15	2.07	1.99	1.95	1.90	1.85	1.80	1.75	1.69
27	4.21	3.35	2.96	2.73	2.57	2.46	2.37	2.31	2.25	2.20	2.13	2.06	1.97	1.93	1.88	1.84	1.79	1.73	1.67
28	4.20	3.34	2.95	2.71	2.56	2.45	2.36	2.29	2.24	2.19	2.12	2.04	1.96	1.91	1.87	1.82	1.77	1.71	1.65
29	4.18	3.33	2.93	2.70	2.55	2.43	2.35	2.28	2.22	2.18	2.10	2.03	1.94	1.90	1.85	1.81	1.75	1.70	1.64
30	4.17	3.32	2.92	2.69	2.53	2.42	2.33	2.27	2.21	2.16	2.09	2.01	1.93	1.89	1.84	1.79	1.74	1.68	1.62
40	4.08	3.23	2.84	2.61	2.45	2.34	2.25	2.18	2.12	2.08	2.00	1.92	1.84	1.79	1.74	1.69	1.64	1.58	1.51
60	4.00	3.15	2.76	2.53	2.37	2.25	2.17	2.10	2.04	1.99	1.92	1.84	1.75	1.70	1.65	1.59	1.53	1.47	1.39
120	3.92	3.07	2.68	2.45	2.29	2.17	2.09	2.02	1.96	1.91	1.83	1.75	1.66	1.61	1.55	1.55	1.43	1.35	1.25
∞	3.84	3.00	2.60	2.37	2.21	2.10	2.01	1.94	1.88	1.83	1.75	1.67	1.57	1.52	1.46	1.39	1.32	1.22	1.00

Degrees of Freedom for the Numerator (ν_1); Degrees of Freedom for the Denominator (ν_2)

T-Test Table

II. Percentage Points of the t Distribution[a]

ν \ α	.40	.25	.10	.05	.025	.01	.005	.0025	.001	.0005
1	.325	1.000	3.078	6.314	12.706	31.821	63.657	127.32	318.31	636.62
2	.289	.816	1.886	2.920	4.303	6.965	9.925	14.089	23.326	31.598
3	.277	.765	1.638	2.353	3.182	4.541	5.841	7.453	10.213	12.924
4	.271	.741	1.533	2.132	2.776	3.747	4.604	5.598	7.173	8.610
5	.267	.727	1.476	2.015	2.571	3.365	4.032	4.773	5.893	6.869
6	.265	.727	1.440	1.943	2.447	3.143	3.707	4.317	5.208	5.959
7	.263	.711	1.415	1.895	2.365	2.998	3.499	4.019	4.785	5.408
8	.262	.706	1.397	1.860	2.306	2.896	3.355	3.833	4.501	5.041
9	.261	.703	1.383	1.833	2.262	2.821	3.250	3.690	4.297	4.781
10	.260	.700	1.372	1.812	2.228	2.764	3.169	3.581	4.144	4.587
11	.260	.697	1.363	1.796	2.201	2.718	3.106	3.497	4.025	4.437
12	.259	.695	1.356	1.782	2.179	2.681	3.055	3.428	3.930	4.318
13	.259	.694	1.350	1.771	2.160	2.650	3.012	3.372	3.852	4.221
14	.258	.692	1.345	1.761	2.145	2.624	2.977	3.326	3.787	4.140
15	.258	.691	1.341	1.753	2.131	2.602	2.947	3.286	3.733	4.073
16	.258	.690	1.337	1.746	2.120	2.583	2.921	3.252	3.686	4.015
17	.257	.689	1.333	1.740	2.110	2.567	2.898	3.222	3.646	3.965
18	.257	.688	1.330	1.734	2.101	2.552	2.878	3.197	3.610	3.922
19	.257	.688	1.328	1.729	2.093	2.539	2.861	3.174	3.579	3.883
20	.257	.687	1.325	1.725	2.086	2.528	2.845	3.153	3.552	3.850
21	.257	.686	1.323	1.721	2.080	2.518	2.831	3.135	3.527	3.819
22	.256	.686	1.321	1.717	2.074	2.508	2.819	3.119	3.505	3.792
23	.256	.685	1.319	1.714	2.069	2.500	2.807	3.104	3.485	3.767
24	.256	.685	1.318	1.711	2.064	2.492	2.797	3.091	3.467	3.745
25	.256	.684	1.316	1.708	2.060	2.485	2.787	3.078	3.450	3.725
26	.256	.684	1.315	1.706	2.056	2.479	2.779	3.067	3.435	3.707
27	.256	.684	1.314	1.703	2.052	2.473	2.771	3.057	3.421	3.690
28	.256	.683	1.313	1.701	2.048	2.467	2.763	3.047	3.408	3.674
29	.256	.683	1.311	1.699	2.045	2.462	2.756	3.038	3.396	3.659
30	.256	.683	1.310	1.697	2.042	2.457	2.750	3.030	3.385	3.646
40	.255	.681	1.303	1.684	2.021	2.423	2.704	2.971	3.307	3.551
60	.254	.679	1.296	1.671	2.000	2.390	2.660	2.915	3.232	3.460
120	.254	.677	1.289	1.658	1.980	2.358	2.617	2.860	3.160	3.373
∞	.253	.674	1.282	1.645	1.960	2.326	2.576	2.807	3.090	3.291

ν = degrees of freedom.

[a] Adapted with permission from *Biometrika Tables for Statisticians*, Vol. 1, 3rd edition, by E. S. Pearson and H. O. Hartley, Cambridge University Press, Cambridge, 1966.

Surface-Down Cracking (Longitudinal Cracking)

t-Test: Paired Two Sample for Means

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	24400	19799
Variance	132494380	89069710
Observations	240	240
df	239	
t Stat	33.22	
P(T<=t) two-tail	1.55E-91	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	24400	22865
Variance	132494380	118267322
Observations	240	240
df	239	
t Stat	35.36	
P(T<=t) two-tail	6.33E-97	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	24400	23108
Variance	132494380	120306561
Observations	240	240
df	239	
t Stat	34.88	
P(T<=t) two-tail	9.47E-96	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	24400	22569
Variance	132494380	115036776
Observations	240	240
df	239	
t Stat	34.35	
P(T<=t) two-tail	2.00E-94	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	24400	22894
Variance	132494380	118289620
Observations	240	240
df	239	
t Stat	34.86	
P(T<=t) two-tail	1.09E-95	
t Critical two-tail	1.97	

Bottom-Up Cracking (Alligator)

t-Test: Paired Two Sample for Means

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	11.97	5.30
Variance	30.11	13.36
Observations	240	240
df	239	
t Stat	39.74	
P(T<=t) two-tail	2.74E-107	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	11.97	8.88
Variance	30.11	20.23
Observations	240	240
df	239	
t Stat	32.67	
P(T<=t) two-tail	4.02E-90	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	11.97	10.42
Variance	30.11	23.86
Observations	240	240
df	239	
t Stat	30.52	
P(T<=t) two-tail	2.09E-84	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	11.97	8.92
Variance	30.11	19.39
Observations	240	240
df	239	
t Stat	31.75	
P(T<=t) two-tail	1.02E-87	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	11.97	10.12
Variance	30.11	23.21
Observations	240	240
df	239	
t Stat	30.89	
P(T<=t) two-tail	2.11E-85	
t Critical two-tail	1.97	

Total Pavement Rutting

t-Test: Paired Two Sample for Means

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	15.52	11.64
Variance	9.91	4.08
Observations	240	240
df	239	
t Stat	53.10	
P(T<=t) two-tail	2.68E-134	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	15.52	13.54
Variance	9.91	6.66
Observations	240	240
df	239	
t Stat	53.95	
P(T<=t) two-tail	7.92E-136	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	15.52	14.99
Variance	9.91	9.12
Observations	240	240
df	239	
t Stat	62.26	
P(T<=t) two-tail	1.07E-149	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	15.52	13.81
Variance	9.91	6.76
Observations	240	240
df	239	
t Stat	48.09	
P(T<=t) two-tail	6.72E-125	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	15.52	14.91
Variance	9.91	9.13
Observations	240	240
df	239	
t Stat	73.29	
P(T<=t) two-tail	8.55E-166	
t Critical two-tail	1.97	

International Roughness Index (IRI)

t-Test: Paired Two Sample for Means

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 2: SP19 6% RAS*
Mean	1.82	1.66
Variance	0.07	0.04
Observations	240	240
df	239	
t Stat	41.07	
P(T<=t) two-tail	2.88E-110	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 3: SP19 3% RAS 25% RAP*
Mean	1.82	1.74
Variance	0.07	0.06
Observations	240	240
df	239	
t Stat	40.94	
P(T<=t) two-tail	5.39E-110	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 4: SP12.5 FC1 3% RAS 17% RAP*
Mean	1.82	1.79
Variance	0.07	0.07
Observations	240	240
df	239	
t Stat	37.12	
P(T<=t) two-tail	3.46E-101	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 5: SP12.5 FC2 6% RAS*
Mean	1.82	1.75
Variance	0.07	0.06
Observations	240	240
df	239	
t Stat	36.78	
P(T<=t) two-tail	2.14E-100	
t Critical two-tail	1.97	

	Mix 1: HL3 1.5% RAS 13.5% RAP	*Mix 6: SP12.5 FC2 3% RAS 12% RAP*
Mean	1.82	1.79
Variance	0.07	0.06
Observations	240	240
df	239	
t Stat	38.63	
P(T<=t) two-tail	9.59E-105	
t Critical two-tail	1.97	